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Coastal and Hydraulics Laboratory

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Life-Cycle Analysis of Mid Bay and Poplar Island Projects, Chesapeake Bay, Maryland

Jeffrey A. Melby, Edward F. Thompson, Mary A. Cialone, Jane M. Smith, Leon E. Borgman, Zeki Demirbilek, Jeffrey L. Hanson, and Lihwa Lin

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Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

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10 S. Howard St., Baltimore, MD 21203-1715

ABSTRACT:
This report summarizes the life-cycle design and optimization of structures on three islands in Chesapeake Bay. The islands are Poplar, James, and Barren. The life-cycle analysis is accomplished using a new method termed Empirical Life-Cycle Simulation (ELS). The historical storms selected for simulation include both winter storms (extratropical storms) and hurricanes (tropical storms). Historical water levels due to the combined effect of historical storms and astronomical tides are simulated using a numerical model of the entire Chesapeake Bay. A localized wind-wave growth model is used to hindcast historical waves. The waves are transformed to a number of analysis stations around each island using a separate numerical model. For each analysis location, 148-year time histories of waves and water levels at 3-hour intervals are produced for use in the life-cycle analysis phase of the study. A new empirical time series simulation method for waves and water levels is proposed so that the effects of potential future wave and water level climate can be analyzed. Finally, analysis of storm maximum and extremal analysis of waves and water levels is described for each island. The results of the wave and water level analyses and simulations described above are used to optimize the structure cross sections over the life cycle. Least-cost structural alternatives that also minimize maintenance requirements are proposed based on these investigations.

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Preface

This report describes procedures and results of a life-cycle analysis study of coastal protection structures at Poplar, James, and Barren Islands. The study was performed in support of planning and design studies. The study was performed by the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), for the U.S. Army Engineer District, Baltimore (NAB). The study was conducted during the period March 2004 through June 2005.

Mr. Scott Johnson, NAB, was overall project manager for Chesapeake Bay dredge disposal island studies. Ms. Karen Nook, NAB, was the study manager and point of contact for the Mid Bay studies, including James and Barren Islands. Mr. Greg Bass, NAB, was the study manager and point of contact for the Poplar Island study. Meetings at critical points in the study were the life-cycle analysis planning meeting and site visit on 25 and 26 August 2004, and the review meeting on 8 November 2004 at NAB.

The life-cycle investigation reported herein was conducted by Drs. Jeffrey A. Melby and Edward F. Thompson, both of the Coastal Harbors and Structures Branch (CHSB), CHL. Hydrodynamic modeling was conducted by Ms. Mary A. Cialone, of the Coastal Processes Branch (CPB), CHL; Dr. Zeki Demirbilek, CHSB, and Dr. Lihwa Lin, of the Coastal Engineering Branch, CHL. Wave modeling was conducted by Drs. Jane M. Smith and Jeffrey L. Hanson, CPB. The methodology and computer programs for Empirical Simulation Technique (EST) life-cycle simulation were developed by Dr. Leon E. Borgman of L. E. Borgman, Inc. The final report was edited by Drs. Melby and Thompson. Ms. J. Holley Messing, of the Coastal Engineering Branch, CHL, completed final formatting of the report, and Mr. David Cate was the Information Technology Laboratory editor.

This study was performed under the general supervision of Mr. Thomas W. Richardson, Director, CHL. Direct supervision of this project was provided by Mr. Dennis G. Markle, Chief, CHSB, and Mr. Jose Sanchez, Acting Chief, CHSB.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL James R. Rowan, EN, was Commander and Executive Director.

1 Introduction

Background

A number of project studies for the Chesapeake Bay involving stone revetment design have been conducted over the past decade. The Poplar Island Habitat Restoration project has generated a particularly strong and visible need for design-related studies. Poplar Island is located 25 km (15 miles) south-southeast of Annapolis, MD, along the east side of Chesapeake Bay (Figure 1). The island is a crescent-shaped enhanced habitat with stone revetment around much of its perimeter, about 4.2 km (2.5 miles) long and 0.8 km (0.5 miles) wide. Poplar Island embodies exposure to waves from all directions, with fetch distances ranging from less than 1.7 km (1 mile) to 33 km (20 miles) and more, depending on exposure direction. Poplar Island is also subject to tides and storm surges. The mean tide range is 0.3 m (1.2 ft). Extreme storm surges can reach much higher than the range of even extreme high astronomical tides, adding as much as 2 m (6 ft) to the astronomical tide level.

The occurrence of extreme conditions at Chesapeake Bay island sites involves an interplay between high winds, elevated water levels, high waves, and shallow water depths. The entire bay is exposed to both hurricanes (or tropical storms) and extratropical storms. Structural damage is typically caused by energetic waves directly moving stone on the slope or by high water levels allowing high or moderately high waves to overtop the structure and collapse the crest by undermining the landward side or both. Extreme water levels do not necessarily coincide with extreme wave heights attacking the various reaches of the island perimeter.

Optimized design of Poplar Island and mid-bay island rubble-mound structures presents a complex and difficult challenge. Methods used in past Chesapeake Bay revetment design studies are basically traditional approaches. Recent advances in numerical modeling technology have provided tools for significantly improving the accuracy of wave and water level estimates. With present technology, the time variation of winds, waves, and water levels during historical storms can be hindcast based on available historical information.

Stone size prediction as well as stone damage development is traditionally done, as in the previous studies of Poplar Island, using tools developed for the design of coincident extreme wave and water level during a single storm. The traditional approach has several limitations. First, it requires a careful choice of design wave and water level combinations to be used. Second, it does not account for key life-cycle processes, as when the structure sustains a small amount of damage in a storm, and then in its weakened state, is subjected to another or several more damaging storms. Damage can accumulate over the life of the structure, especially in the interval between repair visits. Third, because the traditional approach does not deal with life-cycle processes, it does not lend itself to clear analysis of the tradeoff between initial construction and maintenance costs over the projected life of the structure, a key economic consideration. Finally, the traditional approach is rooted in historical storm information; it does not take into account the natural variability of future storm conditions.

The *Coastal Engineering Manual* (HQUSACE 2002) is now the current standard for coastal structure design within the U.S. Army Corps of Engineers (USACE). It includes recently updated methods for estimating wave runup and overtopping for design. For compliance with USACE standards, the CEM should be used as a basis for cross-sectional design of island dikes. This includes predicting life-cycle damage to both the armor layer and the toe. The most common technique presently used by the USACE in coastal studies to extend historical storms within a life-cycle or risk analysis utilizes the Empirical Simulation Technique (EST) with historical waves and water levels. The EST method popular within the USACE is actually one of many empirical simulation methods that have been used for decades. EST is superior to other techniques because it is based entirely on historical events and their analysis. Further, it does not presuppose any knowledge of correlation between various parameters that are usually nonlinearly related. Other techniques require that combined probability distributions be determined. Typically, the various parameters are related in a highly nonlinear manner and data do not exist to determine the correlations. Monte-Carlo simulation can produce unrealistic combinations of parameters. The typical EST has been enhanced in this study to incorporate time as a dependent variable and to allow an unlimited number of parameter realizations, which are all correlated. The new modeling technique is termed the Empirical Life-Cycle Simulation (ELS) method.

Because of the limitations in traditional tools and the emergence of improved technology, the U.S. Army Engineer District, Baltimore (hereafter, Baltimore District), requested the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), to develop a state-of-the-art tool for analyzing life-cycle costs of these projects applied to Poplar Island. The Baltimore District also requested that CHL conduct a similar analysis of a planned island project at James Island, along the east side of Chesapeake Bay, 17 miles south of Poplar Island (Figure 1). A third island site in Chesapeake Bay, Barren Island, is under consideration for future protection/restoration efforts. Barren Island is 12 miles south-southeast of James Island, along the east side of Chesapeake Bay opposite the mouth of the Patuxent River (Figure 1). The Baltimore District requested that this island also be included in the analysis. James Island and Barren Island are collectively termed “Mid Bay” sites.

Study Approach

The study described in this report was performed by CHL in support of the Baltimore District’s design efforts at the three island sites: Poplar Island, James Island, and Barren Island. This study had the following goals:

a. Identify historical tropical and extratropical storms needed to develop design conditions at Chesapeake Bay project sites.

b. Acquire wind fields for historical storms identified in *a.*, to be used for water level modeling. Open-ocean winds for most storms are available from previous studies.

c. Adjust wind fields over Chesapeake Bay waters as needed to represent winds over the bay suitable for water level modeling.

Figure 1. Location map of study area

d. Analyze existing historical data from regional anemometers in order to develop local winds over Chesapeake Bay fetches for wave analysis.

e. Compute historical storm water levels using the existing ADCIRC numerical model, updating the regional bathymetry and shoreline grid already developed for other Baltimore District studies at Ocean City Inlet and Assateague Island.

f. Hindcast historical storm waves using model winds along with measured winds from several area anemometers. Compute historical offshore waves using relationships for wind-wave growth over irregular, restricted fetches.

g. Transform waves through shallow nearshore waters to shore using a spectral wave transformation model (STWAVE).

h. Compute responses for these historical events, such as runup, overtopping as a function of crest height, structure damage as a function of stone size, and required toe stone weight. Use techniques based on recommendations given in the CEM.

i. Recreate multiple life cycles of storms and project responses using the ELS method. Each life cycle represents a possible future condition, which is statistically consistent with historical storm forcing, response, and sequencing information. The ELS simulation includes progressive revetment damage caused by successive storms that may occur between maintenance opportunities. Realistic maintenance cycles are incorporated into the simulation.

j. Compute life-cycle damage and function for selected designs that appear to be favorable.

Candidate designs and design evaluation criteria, including environmental considerations, were defined in close coordination with the Baltimore District. Results are summarized based on an analysis of mean and extreme structure responses in multiple life-cycle scenarios developed in *i*. The results will assist the Baltimore District in quantifying design construction cost versus benefit trade-offs between initial construction and expected maintenance.

The historical storms selected for simulation include both winter storms (extratropical storms) and hurricanes (tropical storms). The storms chosen, the reasons for choosing them, and procedures for estimating storm wind and pressure fields are discussed in Chapter 2.

Storm wind and pressure fields over Chesapeake Bay provide the key meteorological forcing that can cause unusually high water levels during storms. To accurately simulate water levels caused by the combined effect of historical storms and astronomical tides, the entire Chesapeake Bay must be modeled. Procedures and results from these hydrodynamic simulations of historical storms are presented in Chapter 3.

Storm winds also generate unusually high waves. Adaptation of winds to local wave growth around the study islands, wave generation, and wave transformation to island shores are described in Chapter 4.

The ELS approach used in this study drew together some important recent advances in statistical procedures for hypothesizing future storm sequences and for predicting cumulative structural responses to a succession of storms. The methodology is described in Chapters 5 and 6.

Life-cycle simulations were developed and summarized for each of the three study islands. Poplar Island results are presented in Chapter 7. Results for James and Barren Islands are given in Chapters 8 and 9.

Conclusions and recommendations are given in Chapter 10. This chapter is followed by the references.

2 Selection of Historical Tropical and Extratropical Storms

This chapter describes the process of selecting 95 historical tropical and extratropical storms for the Mid Bay and Poplar Island projects. The hydrodynamic model ADCIRC (Luettich et al. 1992) was applied to the Chesapeake Bay area for each historical event, which is documented in Chapter 3. The simulations were performed for the 95 historical storms to report predicted water levels at the three island locations described in Chapter 1. Predicted water levels at the three islands were extracted for each of the storm simulations to be applied in the life-cycle analysis and to be applied in the wave-modeling task.

The purpose of the simulations presented in Chapter 3 was to determine water levels under various storm conditions at the three island sites. Tasks accomplished to attain the goal included: (a) identifying historical tropical and extratropical storms that passed through the Chesapeake Bay region, (b) acquiring wind fields for historical storms identified as potential storms to simulate, (c) adjusting wind fields over land and over bay as needed to represent overland wind adjustments and over-bay wind adjustments, (d) analyzing existing historical data from regional anemometers to determine local winds over Chesapeake Bay, (e) developing a numerical finite element grid of Chesapeake Bay, including overland areas, (f) validating the hydrodynamic model ADCIRC to several historical storm events, (g) applying ADCIRC to the suite of historical storm events to compute storm water levels, and (h) extracting water levels at the three island sites. This chapter documents the completion of tasks a-d.

Selection of Storms

Hurricanes

The North Atlantic Hurricane Track Database (1851-2003) was extracted from the Internet web site (http://weather.unisys.com/hurricane) to determine the set of tropical storms that traversed the Chesapeake Bay region. Fifty-two hurricanes (Table 1) were selected from the database for simulation based on the following criteria: storms with maximum wind speeds greater than 50 knots in the area between lat. 36 and 39°N and long. 75 and 79°W.

Table 1
Selected Tropical Storms (Hurricanes)

Simulation	Year/Month	Name	Database Number
1	1856/August	None	31
2	1861/September	None	64
3	1861/November	None	67
4	1863/September	None	78
5	1874/September	None	156
6	1876/September	None	165
7	1877/September	None	172
8	1878/October	None	187
9	1879/August	None	190
10	1880/September	None	202
11	1881/September	None	213
12	1888/October	None	269
13	1889/September	None	277
14	1893/June	None	302
15	1893/August	None	307
16	1893/September	None	310
17	1893/October	None	312
18	1894/September	None	316
19	1894/October	None	317
20	1897/October	None	336
21	1899/August	None	347
22	1899/October	None	351
23	1904/September	None	384
24	1908/July	None	409
25	1923/October	None	492
26	1933/August	None	562
27	1933/September	None	567
28	1935/August	None	588
29	1936/September	None	605
30	1944/July	None	667
31	1944/September	None	671
32	1946/July	None	688
33	1953/August	Barbara	755
34	1954/October	Hazel	776
35	1955/August	Connie	780
36	1955/August	Diane	781
37	1955/September	Ione	787
38	1960/July	Brenda	830
39	1960/August	Donna	832

(Continued)

Table 1 (Concluded)			
Simulation	**Year/Month**	**Name**	**Database Number**
40	1967/September	Doria	892
41	1971/August	Doria	937
42	1981/June	Bret	1030
43	1983/September	Dean	1050
44	1985/September	Gloria	1070
45	1986/August	Charley	1077
46	1992/September	Danielle	1137
47	1996/July	Bertha	1175
48	1996/August	Fran	1179
49	1998/August	Bonnie	1196
50	1998/August	Earl	1199
51	1999/September	Floyd	1214
52	2003/September	Isabel	1264

Figure 2 shows storm tracks for the 52 hurricanes. The database contained the maximum wind speed and minimum pressure as each storm tracked across the Atlantic Ocean and/or Gulf of Mexico. Wind and pressure fields were generated for a given track using the Planetary Boundary Layer (PBL) model (Cardone 1977). Adjustments for over land and over bay were made to the wind fields as described below. The wind and pressure fields were then applied in the ADCIRC model simulations for Chesapeake Bay to obtain the response of the bay to each storm.

Northeasters

Forty-three northeasters (1954-2003) were identified in the Atmospheric Environmental Service of Canada (AES-40) wind fields (Swail et al. 2000) and in the reanalysis project database (Kalnay et al. 1996) by the U.S. National Centers for Environmental Prediction (NCEP) and the National Center for Atmospheric Research (NCAR) (Table 2). Pressure fields were obtained from the NCEP/NCAR database. Storms were selected based on the criteria of peak wind speeds greater than 20 m/sec (66 ft/sec) or 10 m/sec (33 ft/sec) with durations exceeding 3 days at the ocean entrance of the Chesapeake Bay. Figure 3 shows an example of selecting northeasters based on time series of wind speed and direction extracted from AES-40 for 1999 at the bay entrance. Wind speeds above 10 m/sec (33 ft/sec) are shown as black crosses. Northeaster storms are identified with green circles and northwesters are identified with blue circles. Hurricane wind speeds are identified with magenta circles. Adjustments for over land and over bay were made to the wind fields as described below. The wind and pressure fields were then applied in the ADCIRC model simulations for Chesapeake Bay to attain the response of the bay to each storm.

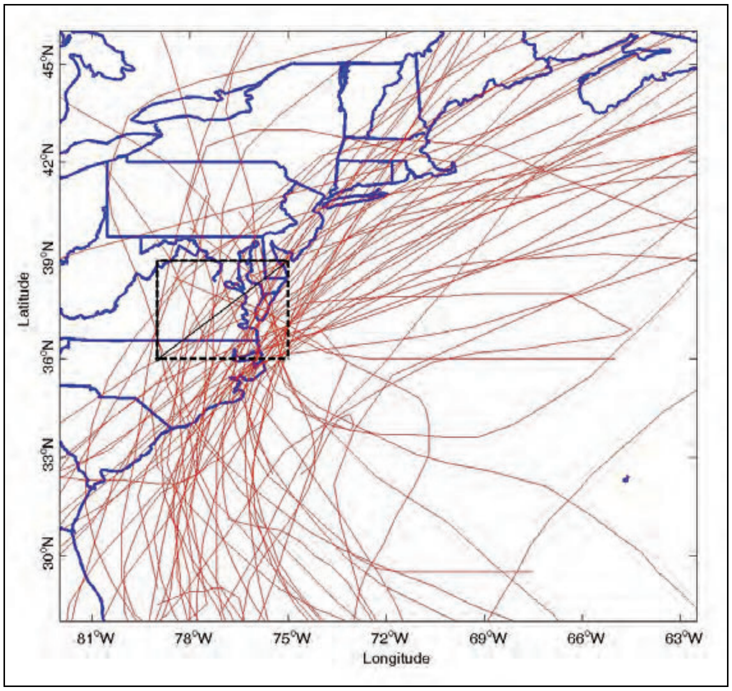


Figure 2. Tracks of 52 hurricanes (1851-2003) and rectangle window (dash-line) used for storm selection

Adjustments to Wind and Water Levels

Water level data

NOAA historical water level data (1996-2003) for Chesapeake Bay were extracted from the Internet web site (http://co-ops.nos.noaa.gov/data_res.html) to examine seasonal and daily water level variations, excluding daily tides. These variations were applied to the model results to account for the monthly mean variation of the water level.

Over-land wind adjustment

AES-40, NCEP/NCAR, and PBL model wind fields are generally accurate for the open coast and ocean applications. In the Chesapeake Bay and adjacent land area, the wind fields needed to be adjusted for over-bay and over-land effects. This was done using the equation:

$$U_L = U_W/R_L \qquad (1)$$

where U_L is the wind speed over land, U_W is the wind speed over water, and R_L is an adjustment factor, and following procedures described in Part II of the *Coastal Engineering Manual* (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 2002). Figure 4 shows an example comparing AES-40 winds with and without the over-land adjustment with measured data at NOAA sta 8577330 (38°19′00″N, 76°27′12″W) for 8-15 September 2003, during the passage of a northeaster system. The necessity of adjustment for the over-land effect to AES-40 winds is evident.

Table 2
Selected Extratropical Storms (Northeasters)

Simulation	Year/Month/Date/Time	Duration, days	Mean Wind Speed, m/sec (ft/sec)
1	1954/01/21/12	2.5	18.4 (60.4)
2	1956/10/16/06	3.5	11.7 (38.4)
3	1956/10/24/06	6.5	14.3 (46.9)
4	1957/10/02/06	4.0	13.7 (44.9)
5	1958/02/15/12	6.0	14.9 (48.9)
6	1958/10/19/12	3.0	16.7 (54.8)
7	1962/03/05/06	3.0	16.3 (53.5)
8	1962/11/26/00	9.5	14.5 (47.6)
9	1966/01/26/06	6.0	15.8 (51.8)
10	1969/01/19/18	3.0	12.5 (41.0)
11	1972/05/24/00	4.0	14.0 (45.9)
12	1972/10/04/06	4.5	13.0 (42.7)
13	1974/11/30/18	4.5	14.6 (47.9)
14	1975/06/28/18	3.5	14.8 (48.6)
15	1977/10/29/00	5.0	12.4 (40.7)
16	1978/04/26/00	2.5	14.7 (48.2)
17	1980/12/26/18	5.0	13.2 (43.3)
18	1981/08/19/00	4.5	12.3 (40.4)
19	1983/02/10/18	5.0	13.4 (44.0)
20	1884/03/28/12	3.0	15.8 (51.8)
21	1884/09/26/12	6.0	13.1 (43.0)
22	1884/10/10/12	4.5	14.8 (48.6)
23	1884/11/19/06	3.5	13.0 (42.7)
24	1985/10/28/12	9.0	13.6 (44.6)
25	1986/11/29/18	4.5	12.8 (42.0)
26	1987/02/15/00	3.5	12.8 (42.0)
27	1988/04/11/12	3.0	14.8 (48.6)
28	1989/03/07/06	4.0	13.6 (44.6)
29	1991/01/07/00	5.0	13.4 (44.0)
30	1991/04/18/00	3.5	14.4 (47.2)
31	1991/10/28/00	4.0	14.6 (47.9)
32	1991/11/08/00	2.5	18.2 (59.7)
33	1993/03/12/12	3.0	13.8 (45.3)
34	1994/10/12/00	4.5	13.1 (43.0)
35	1996/10/03/12	6.5	12.4 (40.7)
36	1997/06/01/00	7.0	12.0 (39.4)
37	1997/10/14/06	7.0	12.1 (39.7)
38	1998/05/10/12	4.5	12.2 (40.0)
39	1999/04/28/12	6.0	12.5 (41.0)
40	1999/08/29/12	8.5	14.2 (46.6)
41	2000/05/28/12	3.5	15.0 (49.2)
42	2003 /04/08/00	4.5	12.1 (39.7)
43	2003/09/08/06	4.5	13.9 (45.6)

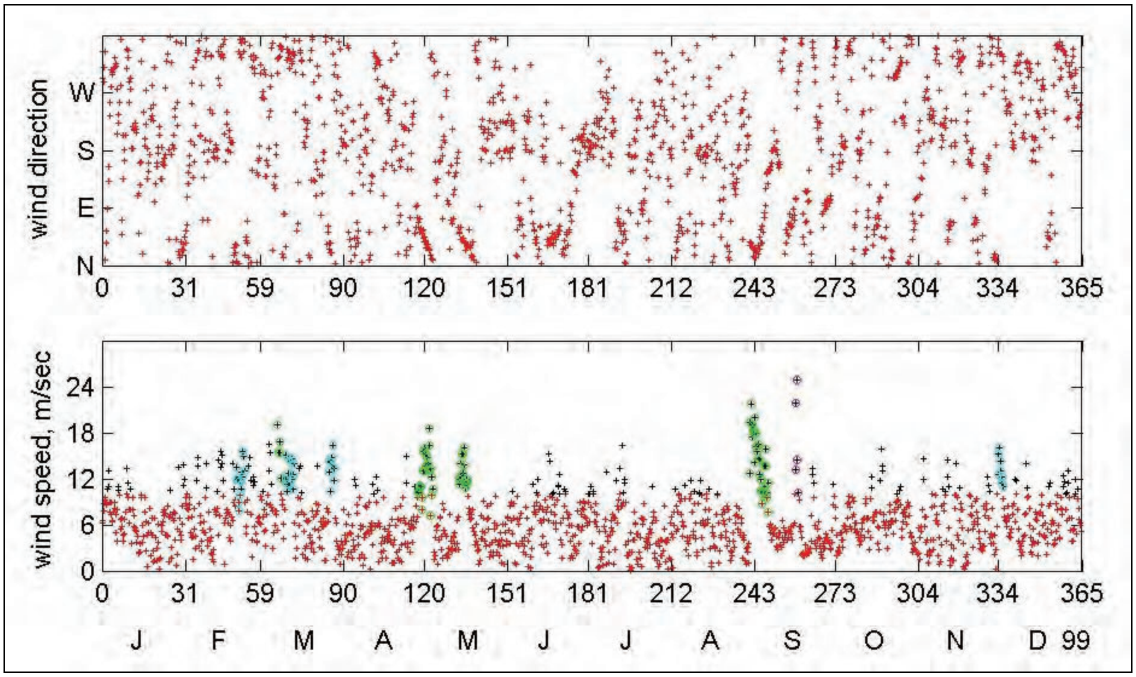


Figure 3. Screened wind events for northeasters (green circles), northwesters (blue circles), and hurricane wind speed (magenta circles) for 1999

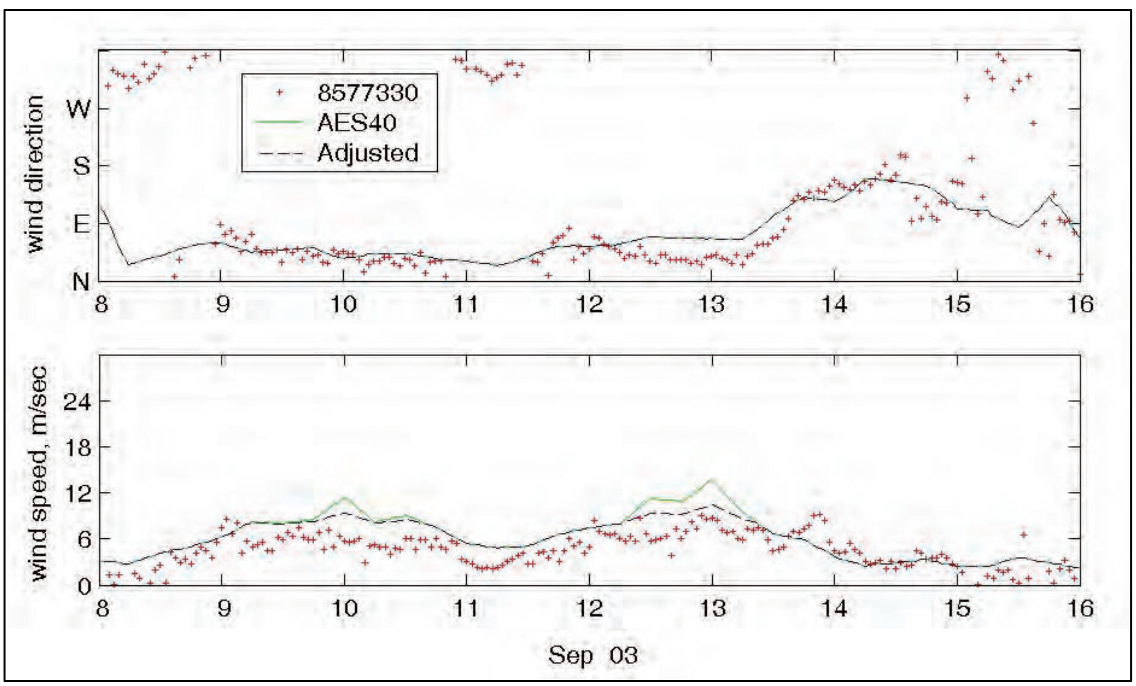


Figure 4. Comparison of AES-40 winds with (dash line) and without (solid line) over-land adjustment with measured data (crosses) at NOAA sta 8577330 for 8-15 September 2003

3 Hydrodynamic Modeling of Historical Tropical and Extratropical Storms

This chapter describes numerical simulations of 95 historical tropical and extratropical storms for the Mid Bay and Poplar Island project. The hydrodynamic model ADCIRC (Luettich et al. 1992) was applied to the Chesapeake Bay area for each historical event. The simulations were performed for the 95 historical storms to report predicted water levels at the three island locations described in Chapter 1. Predicted water levels at the three islands were extracted for each of the storm simulations to be applied in the life-cycle analysis and to be applied in the wave modeling task.

The purpose of the simulations presented in this chapter was to determine water levels under various storm conditions at the three island sites. Tasks accomplished to attain the goal included: (a) identifying historical tropical and extratropical storms that passed through the Chesapeake Bay region, (b) acquiring wind fields for historical storms identified as potential storms to simulate, (c) adjusting wind fields over land and over bay as needed to represent overland wind adjustments and over-bay wind adjustments, (d) analyzing existing historical data from regional anemometers to determine local winds over Chesapeake Bay, (e) developing a numerical finite element grid of Chesapeake Bay, including overland areas, (f) validating the hydrodynamic model ADCIRC to several historical storm events, (g) applying ADCIRC to the suite of historical storm events to compute storm water levels, and (h) extracting water levels at the three island sites. This chapter documents the completion of tasks e-h.

Numerical Model

ADCIRC is documented in technical reports and technical notes, as well as in the literature of study applications and engineering projects. A short description of the model is given here to provide a general understanding of the function of the model. For more details the reader is referred to the references provided in the sections below.

ADCIRC is a highly developed numerical model for solving the equations of motion for a moving fluid on a rotating earth (Luettich et al. 1992). It serves as the primary Corps of Engineers' regional oceanographic and storm surge model and is certified by the Federal Emergency Management Agency. The equations are formulated with hydrostatic pressure and Boussinesq approximations and are discretized in space with the finite-element method and in time with the finite-difference method. ADCIRC can be run either as a two-dimensional (2D) depth-integrated (2DDI) model or as a three-dimensional (3D) model. Water surface elevation is obtained from the solution of the depth-integrated continuity equation in the Generalized Wave-Continuity Equation (GWCE) form. Flow velocity is obtained from the solution of either the 2DDI or 3D momentum equations. All nonlinear terms are retained in these equations.

ADCIRC can be operated in either a Cartesian or a spherical coordinate system. ADCIRC boundary conditions include specified elevation (harmonic tidal constituents or time series), specified normal flow (harmonic tidal constituents or time series), zero normal flow, slip or no-slip conditions for velocity, external barrier overflow out of the domain, internal barrier overflow between sections of the domain, surface stress (wind and/or wave radiation stress), atmospheric pressure, and outward radiation of waves (Sommerfield condition). ADCIRC can be forced with elevation, normal flow, or surface stress boundary conditions, tidal potential, and earth load/self attraction tide. Recently, global-scale ADCIRC studies were completed on high-performance computers to provide accurate tidal constituents for the Atlantic Ocean, Gulf of Mexico, and Pacific Ocean coasts of the United States to furnish reliable tidal constituents for project-scale simulations (Mukai et al. 2002; Spargo et al. 2004).

Numerical Grid Development

A regional-scale ADCIRC grid with a rudimentary representation of Chesapeake Bay was developed through previous Coastal Inlets Research Program (CIRP) and Offshore and Coastal Technologies, Inc. (OCTI) studies (Figure 5). This grid was refined in Chesapeake Bay and far-field areas for the present study (Figure 6) using National Ocean Service (NOS) Digital Navigation Charts (DNC). In this hydrodynamic study, the existing-condition bathymetry was taken from a composite from the Virginia Institute of Marine Science (VIMS), Geophysical Data Management System (GEODAS). Periodic surveys conducted by U.S. Army Engineer District, Philadelphia, were used to update the topography for the locations of Poplar, James, and Barren Islands relative to the ADCIRC grid mesh. The numerical grid was developed for ADCIRC to represent present-day (2004) conditions. Detailed Chesapeake Bay coastline and bathymetric data were obtained from VIMS and incorporated into the refined ADCIRC grid (Figure 7). Chesapeake and Delaware Canal bathymetric data were obtained from the U.S. Army Engineer Division, North Atlantic. Further grid development included the incorporation of overbank areas into the Chesapeake Bay tributaries to accurately predict storm surge in these relatively narrow branches of the bay (Figure 8). The ADCIRC grid was extended to include low-land topography data to +10 m (33 ft), mean tide level, from the U.S. Geological Survey Digital EEM database GTOPO30 - 30-sec arc resolution (http://edcdaac.usgs.gov/gtopo30/gtopo30.asp). The grid was constructed with a minimum resolution of 50 m (164 ft) and a maximum cell size of 500 m (1,640 ft) in the open ocean. The ADCIRC grid generated in this process was applied to tidal current and storm surge simulations to calculate water levels at the three island sites.

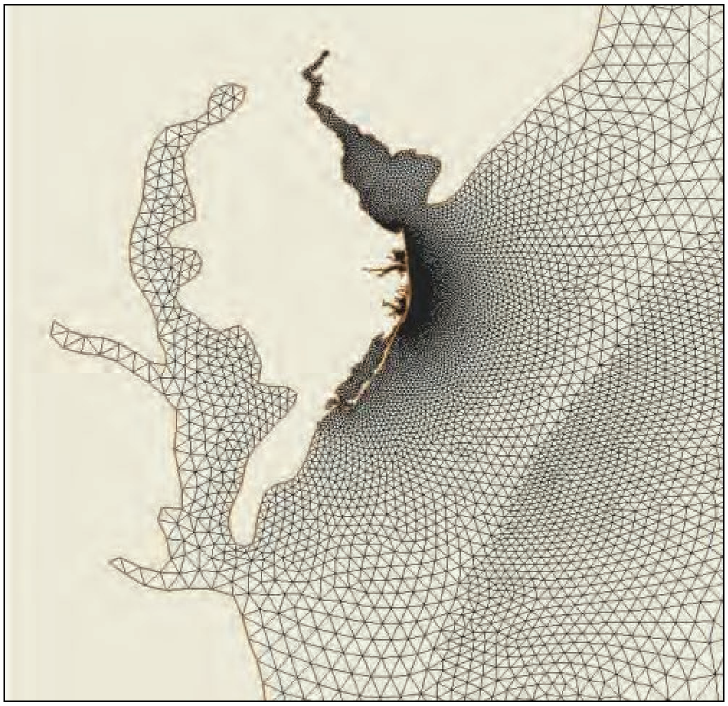


Figure 5. Portion of original ADCIRC grid resolution and shoreline

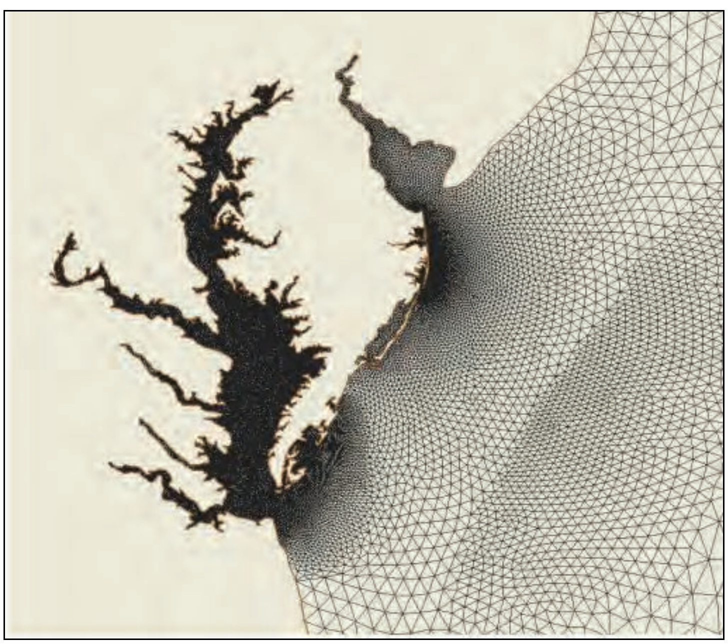


Figure 6. Portion of revised ADCIRC grid resolution and shoreline

Figure 7. Portion of revised ADCIRC grid bathymetry prior to grid extension

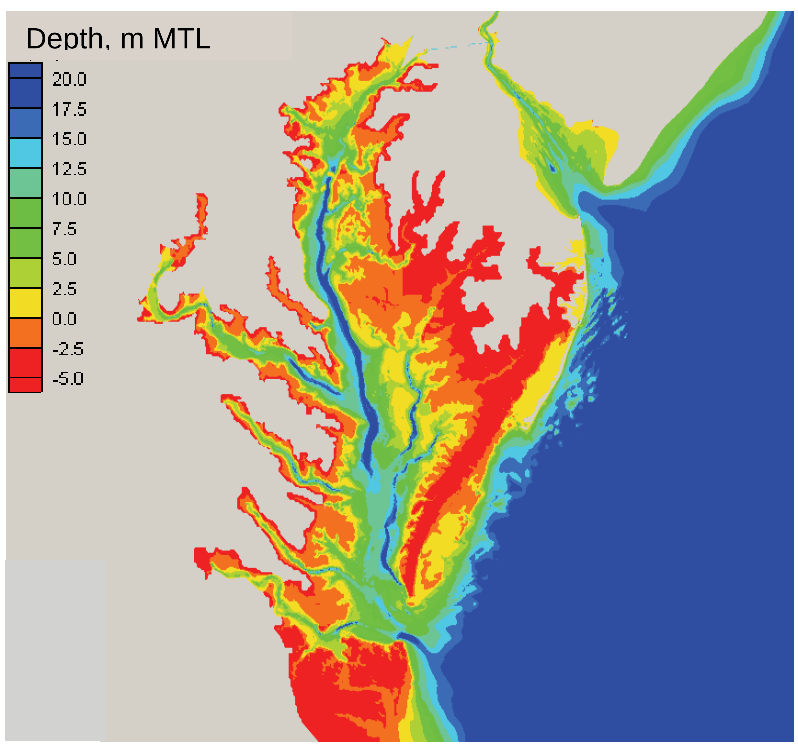


Figure 8. Portion of revised ADCIRC grid bathymetry with overbank extensions

Validation to Storms

NOAA historical water level data (1996-2003) for Chesapeake Bay were extracted from the Internet web site (http://co-ops.nos.noaa.gov/data_res.html) to examine seasonal water level variations and to validate numerical model results. Figure 9, for instance, shows monthly mean water levels at NOAA sta 8574680 (Baltimore, MD) and 8638863 (Bay Bridge, VA) for 2002 and 2003. These figures clearly show the seasonal variation of the mean water level in the bay.

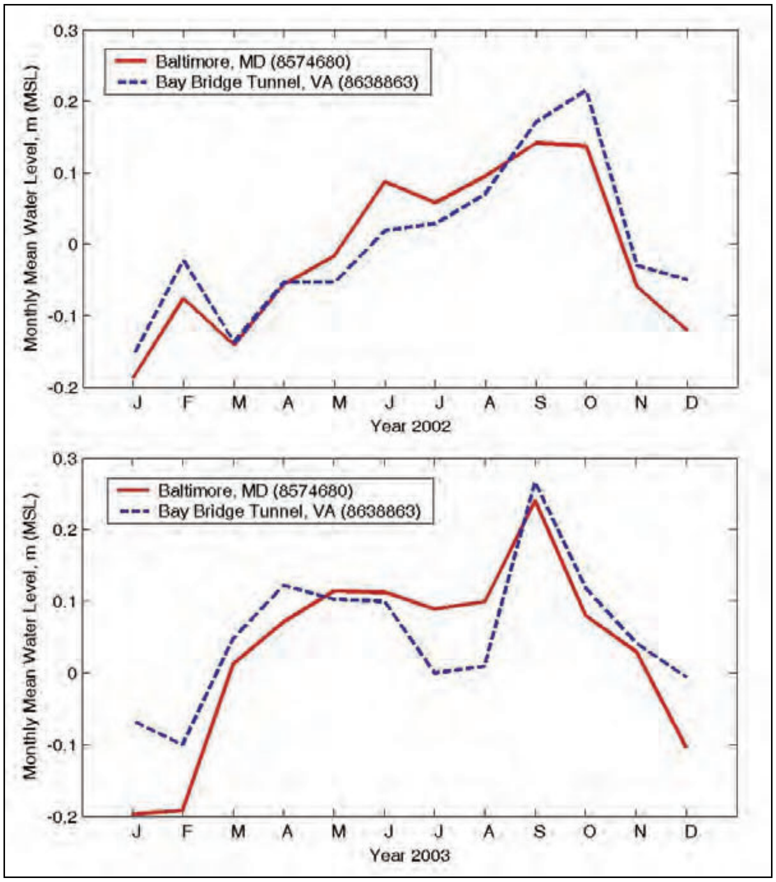


Figure 9. Monthly mean water levels at sta 8574680 and 8638863 for 2002-2003

Tropical storms (hurricanes)

The validation process for tropical storms (hurricanes) applying PBL wind and pressure fields involved comparing water levels at twelve NOAA stations to water levels produced by ADCIRC for two major hurricanes, Fran (1996) and Isabel (2003), and four moderate hurricanes, Bertha (1996), Bonnie (1998), Earl (1998), and Floyd (1999) (Figures 10 and 11, and Table 3). Fran and Isabel approached the bay from the ocean with similar storm tracks nearly perpendicular to the coastline and made the landfall south of the bay. They continued along a northwest course to move farther inland west of the bay. The passage of Bertha was similar to Floyd, as both hurricanes approached and passed the bay paralleling or along the coastline east of the bay. Bonnie and Earl, on the other hand, followed a northeast track from land to ocean crossing the coastline south of the bay. Figure 10 shows storm tracks of these hurricanes. Hurricanes with tracks similar to those of Fran and Isabel can generate higher storm surges as the onshore wind traps more water along the coastline and in the bay.

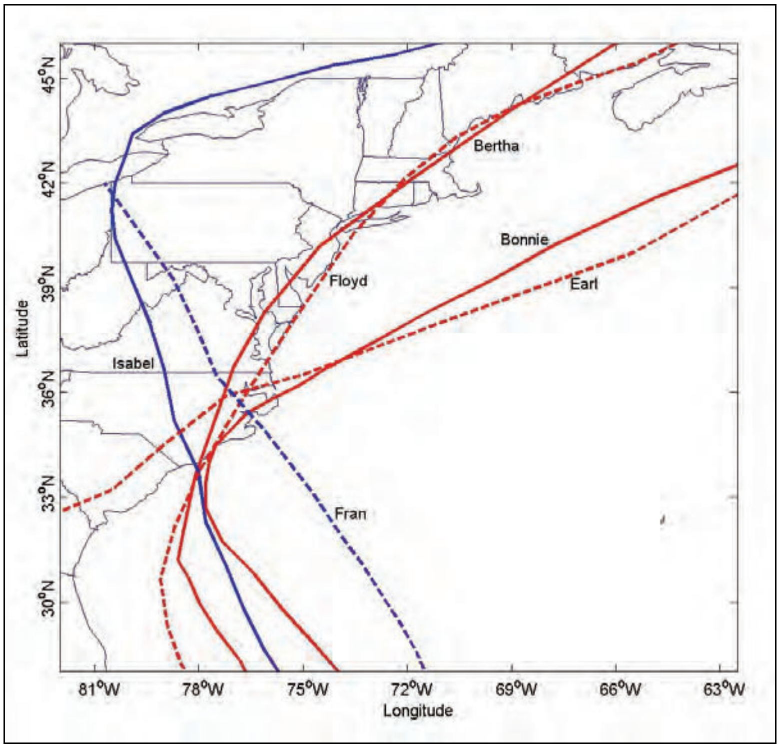


Figure 10. Storm tracks of Hurricanes Bertha, Bonnie, Earl, Floyd, Fran, and Isabel

Table 3
NOAA Stations for Wind/Water Level Measurements (1996-2003), Chesapeake Bay and Delaware Bay

Station No.	Station Name	Coordinates
8551910	Reedy Pt, C&D Canal, DE	39º33'30"N, 75º34'26"W
8557380	Lewes, Ft. Miles, DE	38º46'54"N, 75º07'12"W
8571892	Cambridge, Choptank River, MD	38º34'24"N, 76º04'06"W
8573927	Chesapeake City, MD	39º31'36"N, 75º48'36"W
8574680	Baltimore, MD	38º16'00"N, 76º34'28"W
8575512	U.S. Naval Academy, MD	38º59'00"N, 76º28'48"W
8577330	Solomons Is, MD	38º19'00"N, 76º27'12"W
8632200	Kiptopeke Beach, VA	37º10'00"N, 75º59'18"W
8635750	Lewisetta, Potomac River, VA	37º59'48"N, 76º27'48"W
8636580	Windmill Pt, VA	37º36'42"N, 76º16'30"W
8638610	Sewells Pt, VA	36º56'48"N, 76º19'48"W
8638863	Chesapeake Bay Bridge Tunnel, VA	36º58'00"N, 76º06'48"W

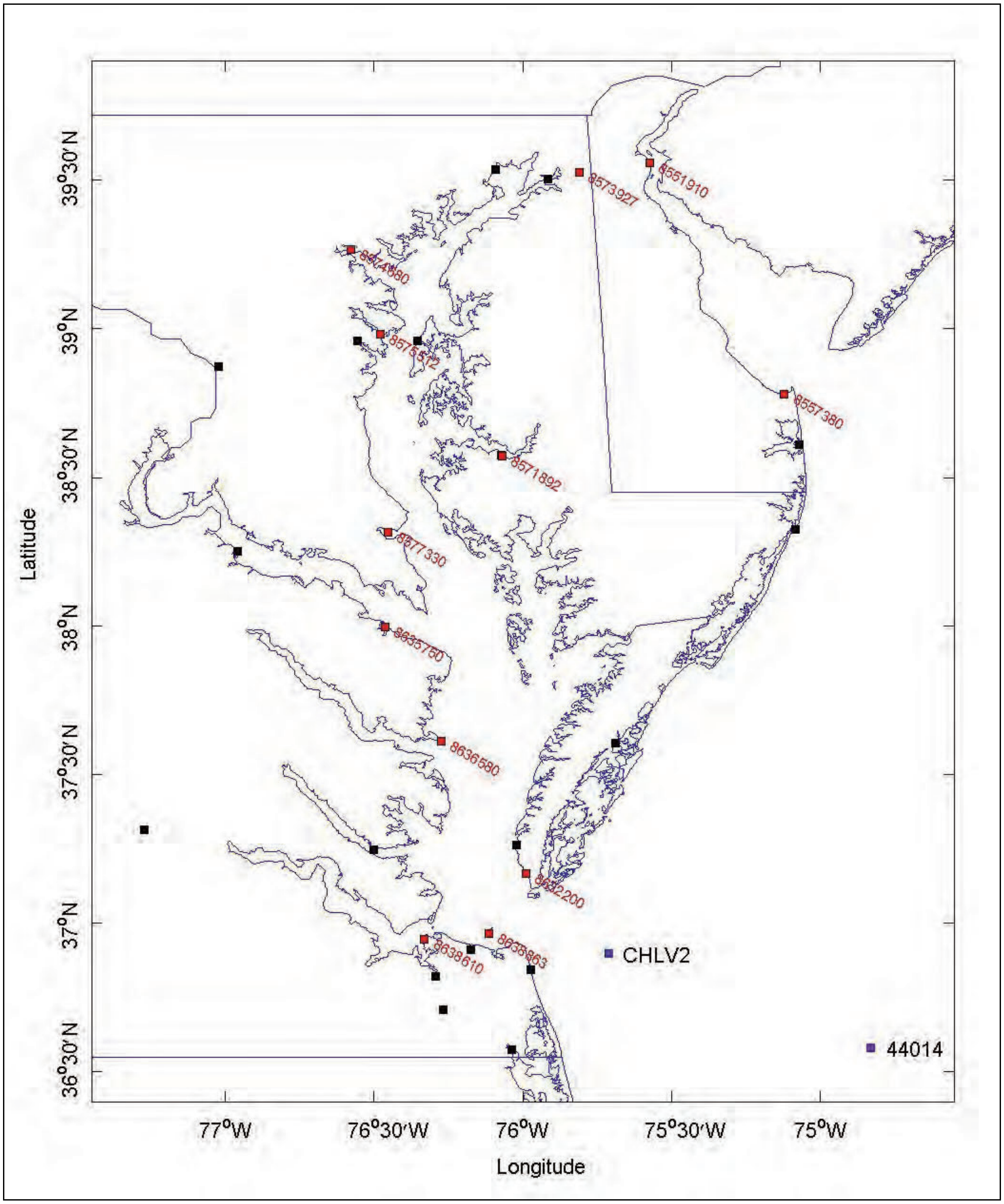


Figure 11. Wind/water level stations, active (red squares) and historical (blue squares)

Figures 12 and 13 show the measured and modeled water level time series at seven NOAA stations for Hurricane Fran. It is noted that an average water level increase of 0.1 m (0.3 ft) in the interval of March to November was added to the model results to account for the seasonal variation. The model results generally agree well with the measured water levels. For instance, at sta 8574680 (Baltimore, MD), near the north end of the bay, both measured and modeled peak water levels are 1.3 m (4.3 ft). At sta 8638863 (Bay Bridge Tunnel, VA), close to the bay entrance, both measured and modeled peak water levels are 0.8 m (2.6 ft). Figures 14 and 15 show the measured and modeled water level time series at seven stations for Hurricane Isabel. The model results again agree well with measured data. At sta 8574680 (Baltimore, MD), measured and modeled peak water levels are 2.2 (7.2 ft) and 2.3 m (7.5 ft), respectively. At sta 8638863 (Bay Bridge Tunnel, VA), both measured and modeled peak water levels are 1.9 m (6.2 ft).

Tables 4-9 compare measured and modeled peak water levels for Hurricanes Bertha, Fran, Bonnie, Earl, Floyd and Isabel. For these hurricanes, the difference of predicted and measured peak water levels ranges from -0.31 to 0.46 m (-1.0 to 1.5 ft). The root-mean-square error of predicted peak water level versus measured data ranges between 0.07 and 0.2 m (0.23 and 0.7 ft). The bias of the predicted peak water level is between -0.1 and 0.31 m (-0.3 and 1.0 ft). Model water levels are generally more reliable, as compared to the measured data, for hurricanes with tracks to Fran and Isabel than those with storm tracks similar to Bertha and Bonnie.

Extratropical storms (northeasters)

The validation process for extratropical (northeaster) storm simulations is the same as for the tropical storms. Figure 16 shows two examples of comparisons of model simulations and measurements for two northeasters. Note the measurements shown for 2003 include a high storm surge on 19 September due to Hurricane Isabel. This event was simulated earlier in the study as part of the model validation for tropical storms. In these examples, an average water level increase of 0.1 m (0.3 ft) in the interval of March to November was added to the model results to account for the seasonal variation. The model water level predictions for the extratropical storms generally agree well with the measured data. For instance, during the extratropical storm in mid-May 1998, at sta 8574680 (Baltimore, MD), the measured and modeled peak water levels are 0.76 and 0.64 m (2.5 and 2.1 ft), respectively. At sta 8638863 (Bay Bridge, VA), the measured and modeled peak water levels are 1.1 and 1.2 m (3.6 and 3.9 ft), respectively. During the extratropical storm around 10 September 2003, at sta 8574680 (Baltimore, MD), the measured and model water levels are 0.53 and 0.46 m (1.7 and 1.5 ft), respectively. At sta 8638863 (Bay Bridge, VA), both measured and modeled peak water levels are 1.0 m (3.1 ft).

The validated ADCIRC model was then applied to the suite of 52 hurricanes and 43 extratropical storms (presented in Chapter 2) to compute water levels at Poplar, James, and Barren Islands to be applied in the wave modeling and life-cycle analysis tasks.

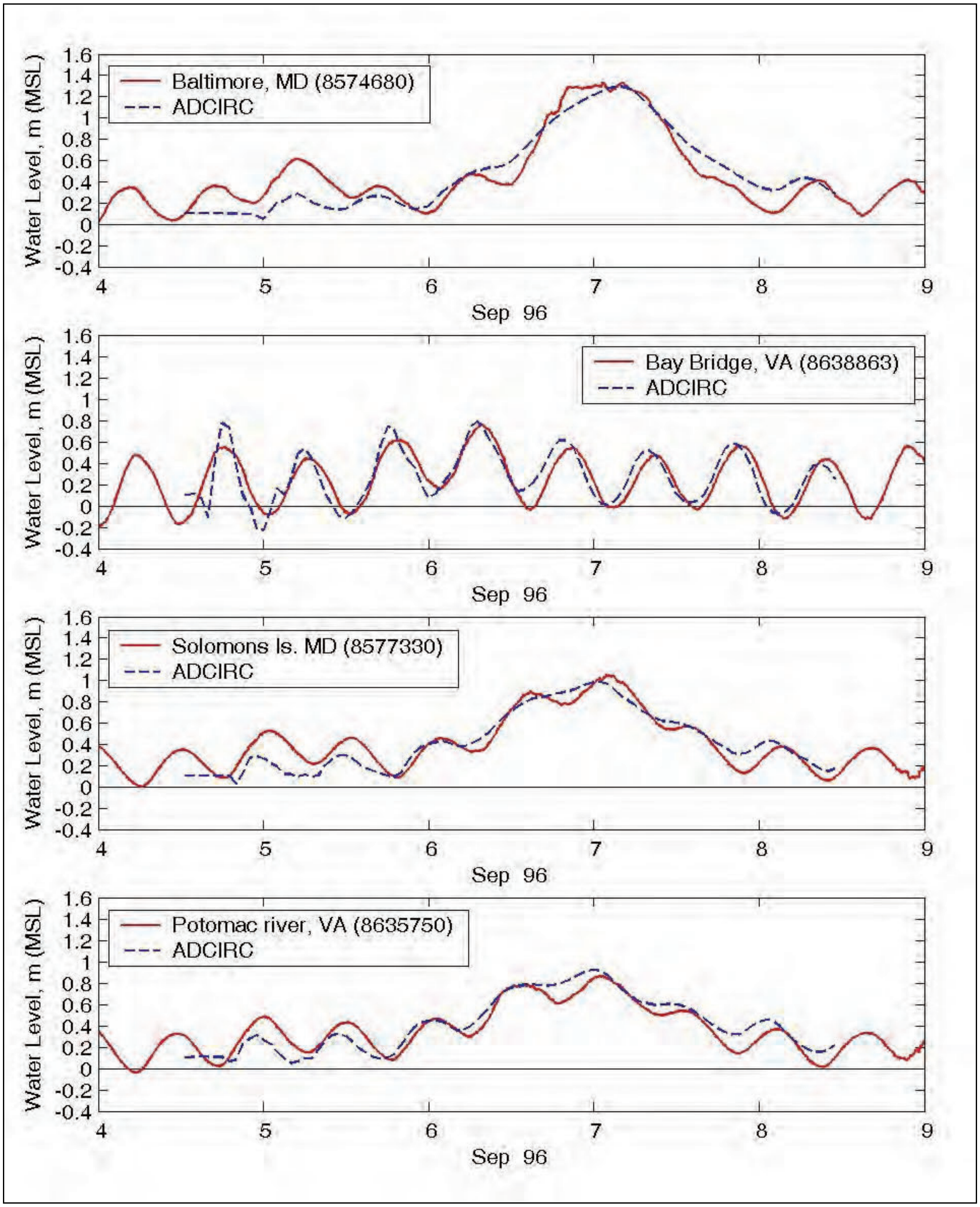


Figure 12. Measured and model water levels at sta 8574680, 8638863, 8577300, and 8635750 for 4-9 September 1996

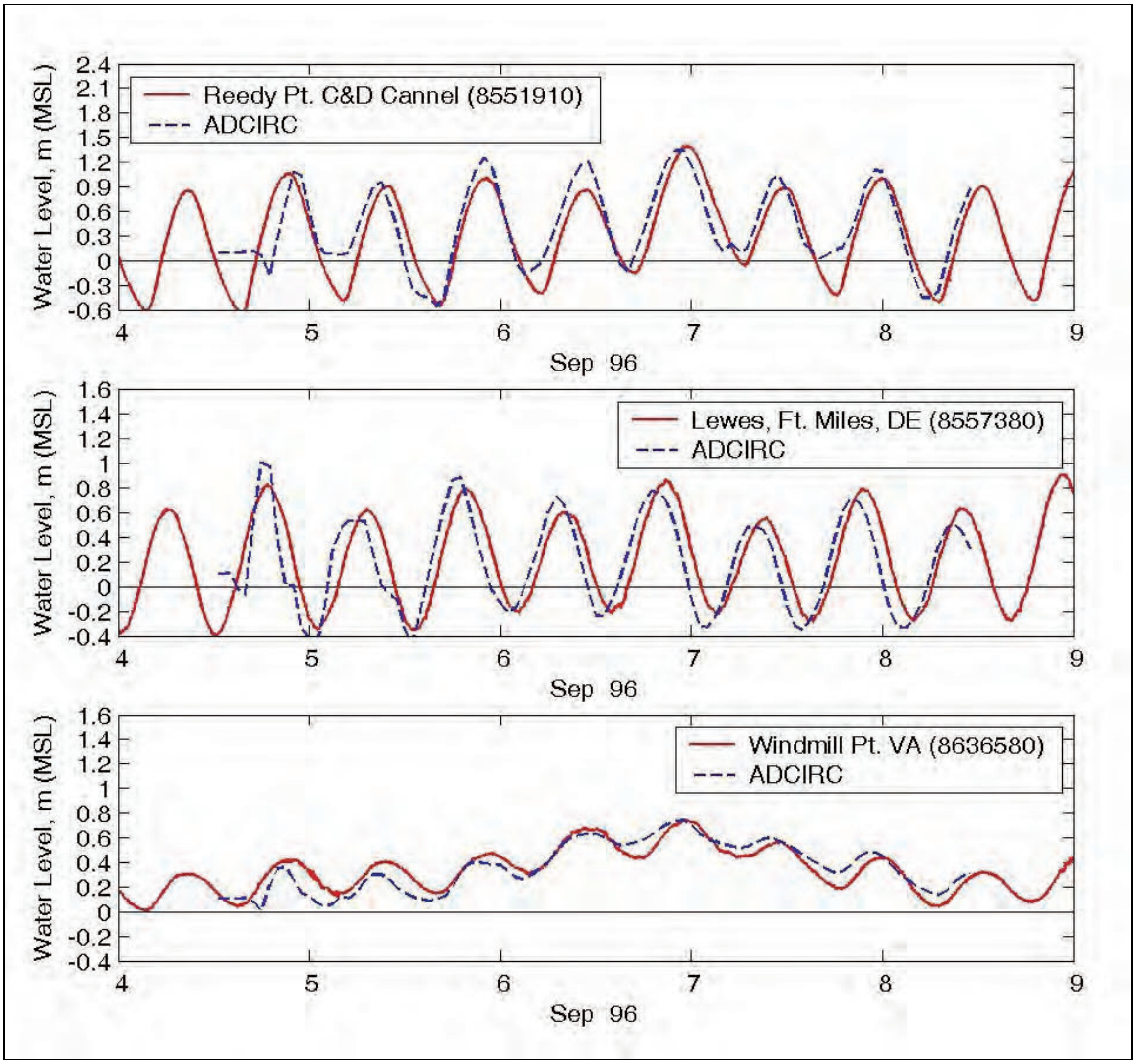


Figure 13. Measured and model water levels at sta 8551910, 8557380, and 8636580 for 4-9 September 1996

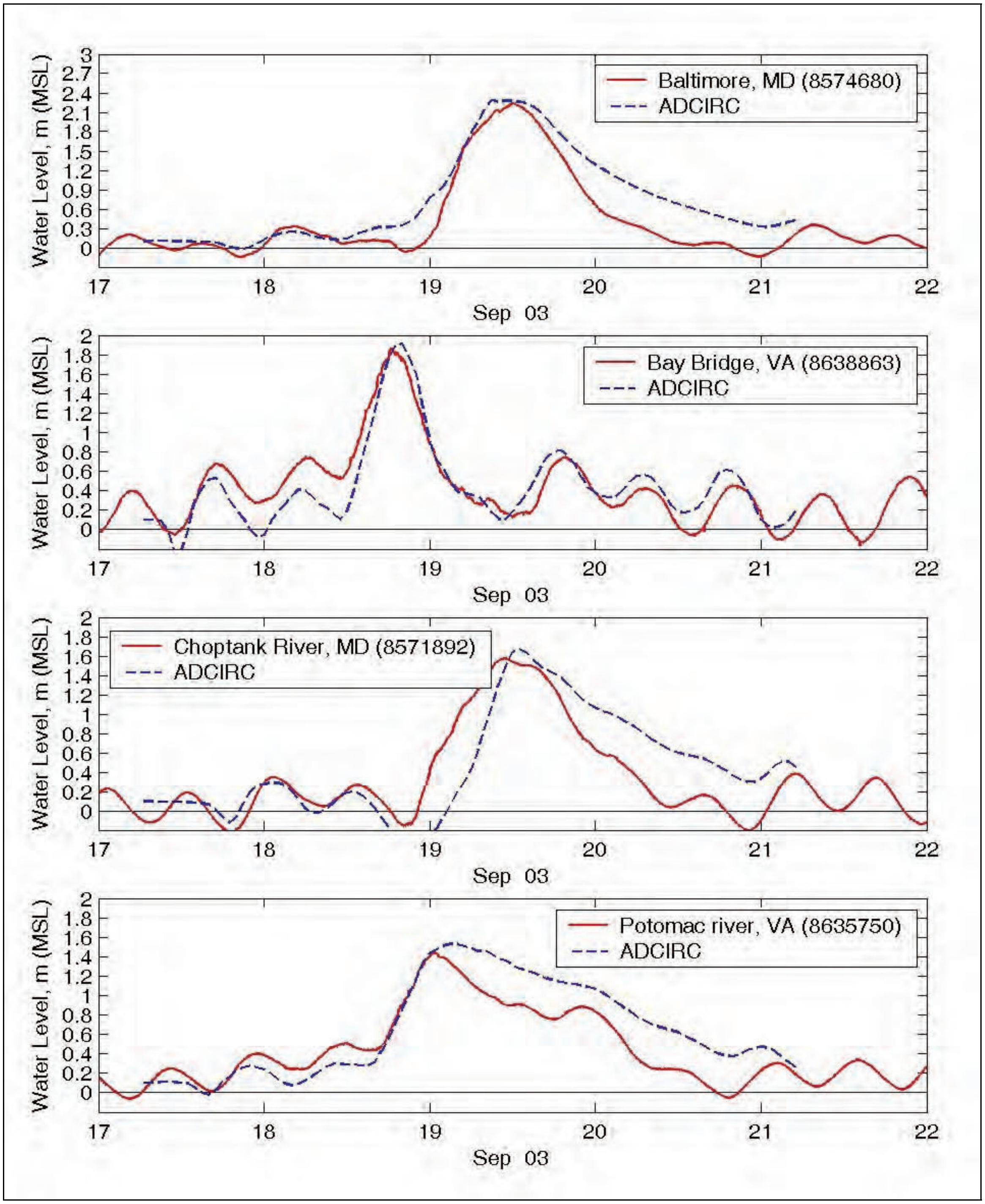


Figure 14. Measured and model water levels at sta 8574680, 8638863, 8571892 and 8635750 for 17-22 September 2003

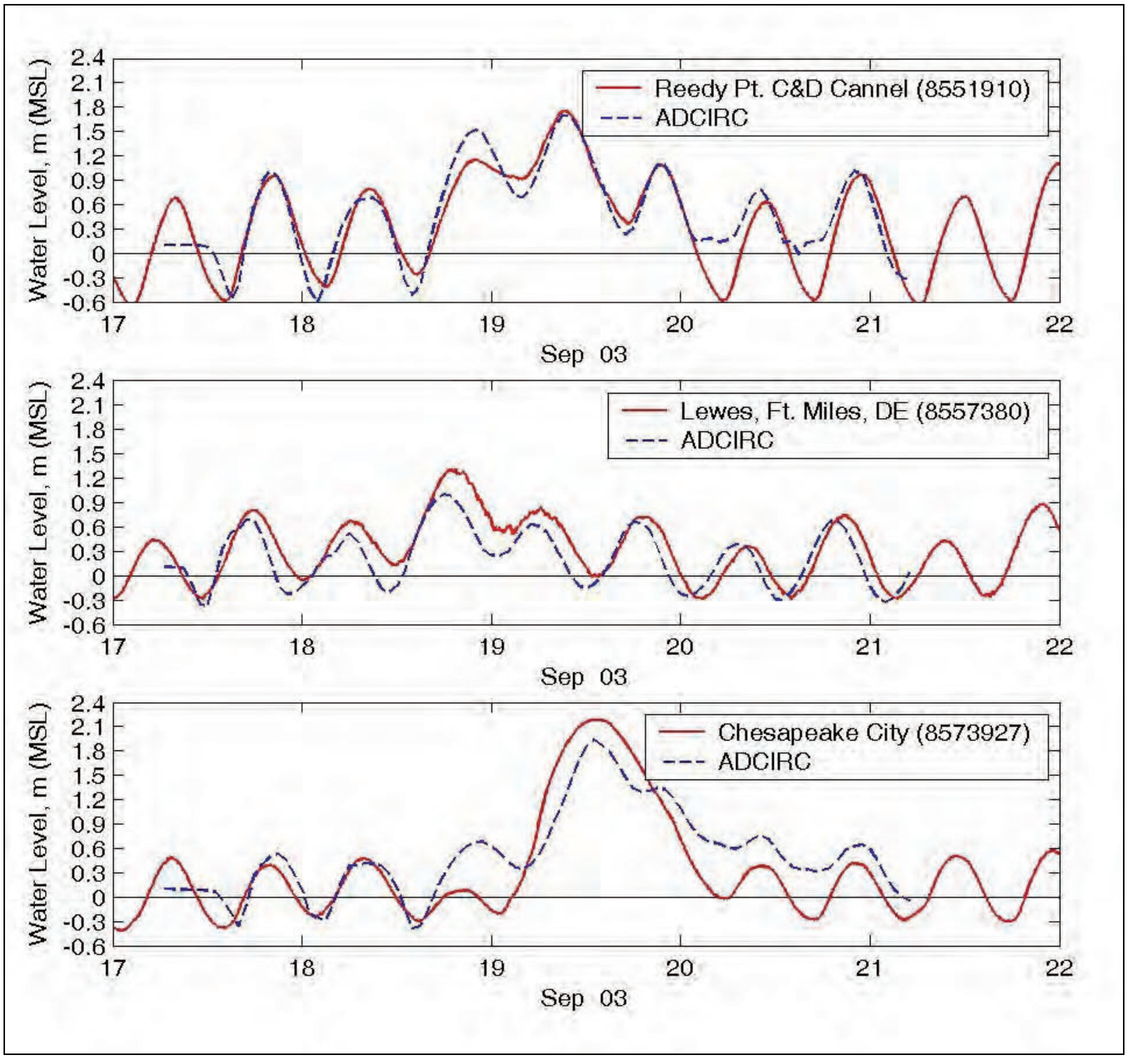


Figure 15. Measured and model water levels at sta 8551910, 8557380, and 8573927 for 17-22 September 2003

Table 4
Comparison of Measured and Predicted Peak Water Levels During Hurricane Bertha (July 1996)

Station No.	Station Name	Measured, m (ft)	Predicted, m (ft)	Predicted–Measured, m (ft)
8551910	Reedy Pt, C&D Canal, DE	1.33 (4.36)	1.37 (4.49)	0.04 (0.13)
8557380	Lewes, Ft. Miles, DE	0.84 (2.76)	0.89 (2.92)	0.05 (0.16)
8574680	Baltimore, MD	0.57 (1.87)	0.70 (2.30)	0.13 (0.43)
8575512	U.S. Naval Academy, MD	0.55 (1.80)	0.78 (2.56)	0.23 (0.75)
8577330	Solomons Is, MD	0.64 (2.10)	0.90 (2.95)	0.26 (0.85)
8632200	Kiptopeke Beach, VA	0.59 (1.94)	0.54 (1.77)	-0.05 (-0.16)
8635750	Lewisetta, Potomac River, VA	0.60 (1.97)	0.83 (2.72)	0.23 (0.75)
8638610	Sewells Pt, VA	0.60 (1.97)	0.57 (1.87)	-0.03 (-0.10)
8638863	Chesapeake Bay Bridge Tunnel, VA	0.60 (1.97)	0.66 (2.17)	0.06 (0.20)

Root-mean-square error of predicted peak water level = 0.11 (m).
Bias = mean of (predicted-measured) = 0.10 (m).

Table 5
Comparison of Measured and Predicted Peak Water Levels During Hurricane Fran (September 1996)

Station No.	Station Name	Measured, m (ft)	Predicted, m (ft)	Predicted-Measured, m (ft)
8551910	Reedy Pt, C&D Canal, DE	1.39 (4.56)	1.34 (4.40)	-0.05 (-0.16)
8557380	Lewes, Ft. Miles, DE	0.86 (2.82)	1.00 (3.28)	0.14 (0.46)
8574680	Baltimore, MD	1.33 (4.36)	1.30 (4.27)	-0.03 (-0.10)
8577330	Solomons Is, MD	1.05 (3.44)	0.99 (3.25)	-0.06 (-0.20)
8635750	Lewisetta, Potomac River, VA	0.87 (2.85)	0.93 (3.05)	0.06 (0.20)
8636580	Windmill Pt, VA	0.74 (2.43)	0.74 (2.43)	0.00 (0.00)
8638863	Chesapeake Bay Bridge Tunnel, VA	0.76 (2.49)	0.79 (2.59)	0.03 (0.10)

Root-mean-square error of predicted peak water level = 0.07 (m).
Bias = mean of (predicted-measured) = 0.01 (m).

Table 6
Comparison of Measured and Predicted Peak Water Levels During Hurricane Bonnie (August 1998)

Station No.	Station Name	Measured, m (ft)	Predicted, m (ft)	Predicted-Measured, m (ft)
8551910	Reedy Pt, C&D Canal, DE	0.85 (2.79)	1.10 (3.61)	0.25 (0.82)
8557380	Lewes, Ft. Miles, DE	0.92 (3.02)	0.76 (2.49)	-0.16 (-0.52)
8571892	Cambridge, Choptank River, MD	0.60 (1.97)	0.62 (2.03)	0.02 (0.07)
8574680	Baltimore, MD	0.62 (2.03)	0.63 (2.07)	0.01 (0.03)
8577330	Solomons Is, MD	0.57 (1.87)	0.67 (2.20)	0.10 (0.33)
8635750	Lewisetta, Potomac River, VA	0.65 (2.13)	0.78 (2.56)	0.13 (0.43)
8636580	Windmill Pt, VA	0.75 (2.46)	0.81 (2.66)	0.06 (0.20)
8638863	Chesapeake Bay Bridge Tunnel, VA	1.23 (4.04)	1.02 (3.35)	-0.21 (-0.69)

Root-mean-square error of predicted peak water level = 0.14 (m).
Bias = mean of (predicted-measured) = 0.03 (m).

Table 7
Comparison of Measured and Predicted Peak Water Levels During Hurricane Earl (September 1998)

Station No.	Station Name	Measured, m (ft)	Predicted, m (ft)	Predicted-Measured, m (ft)
8551910	Reedy Pt, C&D Canal, DE	1.18 (3.87)	1.25 (4.10)	0.07 (0.23)
8557380	Lewes, Ft. Miles, DE	1.13 (3.71)	0.89 (2.92)	-0.24 (-0.79)
8571892	Cambridge, Choptank River, MD	0.64 (2.10)	0.54 (1.77)	-0.10 (-0.33)
8574680	Baltimore, MD	0.58 (1.90)	0.44 (1.44)	-0.14 (-0.46)
8577330	Solomons Is, MD	0.54 (1.77)	0.42 (1.38)	-0.12 (-0.39)
8635750	Lewisetta, Potomac River, VA	0.53 (1.74)	0.44 (1.44)	-0.09 (-0.30)
8636580	Windmill Pt, VA	0.51 (1.67)	0.45 (1.48)	-0.06 (-0.20)
8638863	Chesapeake Bay Bridge Tunnel, VA	0.81 (2.66)	0.72 (2.36)	-0.09 (-0.30)

Root-mean-square error of predicted peak water level = 0.08 (m).
Bias = mean of (predicted – measured) = -0.10 (m).

Table 8
Comparison of Measured and Predicted Peak Water Levels During Hurricane Floyd (September 1999)

Station No.	Station Name	Measured, m (ft)	Predicted, m (ft)	Predicted-Measured, m (ft)
8551910	Reedy Pt, C&D Canal, DE	1.31 (4.30)	1.56 (5.12)	0.25 (0.82)
8557380	Lewes, Ft. Miles, DE	1.27 (4.17)	1.40 (4.59)	0.13 (0.43)
8571892	Cambridge, Choptank River, MD	0.66 (2.17)	1.11 (3.64)	0.45 (1.48)
8574680	Baltimore, MD	0.62 (2.03)	1.06 (3.48)	0.44 (1.44)
8577330	Solomons Is, MD	0.65 (2.13)	1.11 (3.64)	0.46 (1.51)
8635750	Lewisetta, Potomac River, VA	0.85 (2.79)	1.26 (4.13)	0.41 (1.35)
8636580	Windmill Pt, VA	0.85 (2.79)	1.16 (3.81)	0.31 (1.02)
8638863	Chesapeake Bay Bridge Tunnel, VA	1.30 (4.27)	1.33 (4.36)	0.03 (0.10)

Root-mean-square error of predicted peak water level = 0.15 (m).
Bias = mean of (predicted – measured) = 0.31 (m).

Table 9
Comparison of Measured and Predicted Peak Water Levels During Hurricane Isabel (September 2003)

Station No.	Station Name	Measured, m (ft)	Predicted, m (ft)	Predicted-Measured, m (ft)
8551910	Reedy Pt, C&D Canal, DE	1.75 (5.74)	1.69 (5.54)	-0.06 (-0.20)
8557380	Lewes, Ft. Miles, DE	1.31 (4.30)	1.00 (3.28)	-0.31 (-1.02)
8571892	Cambridge, Choptank River, MD	1.58 (5.18)	1.68 (5.51)	0.10 (0.33)
8573927	Chesapeake City, MD	2.18 (7.15)	1.94 (6.36)	-0.26 (-0.85)
8574680	Baltimore, MD	2.24 (7.35)	2.28 (7.48)	0.04 (0.13)
8575512	U.S. Naval Academy, MD	1.98 (6.50)	2.30 (7.55)	0.32 (1.05)
8577330	Solomons Is, MD	1.85 (6.07)	1.80 (5.91)	-0.05 (-0.16)
8632200	Kiptopeke Beach, VA	1.55 (5.09)	1.70 (5.58)	0.15 (0.49)
8635750	Lewisetta, Potomac River, VA	1.44 (4.72)	1.53 (5.02)	0.09 (0.30)
8636580	Windmill Pt, VA	1.48 (4.86)	1.30 (4.27)	-0.18 (-0.59)
8638610	Sewells Pt, VA	1.99 (6.53)	2.35 (7.71)	0.36 (1.18)
8638863	Chesapeake Bay Bridge Tunnel, VA	1.87 (6.14)	1.91 (6.27)	0.04 (0.13)

Root-mean-square error of predicted peak water level = 0.20 (m).
Bias = mean of (predicted – measured) = 0.02 (m).

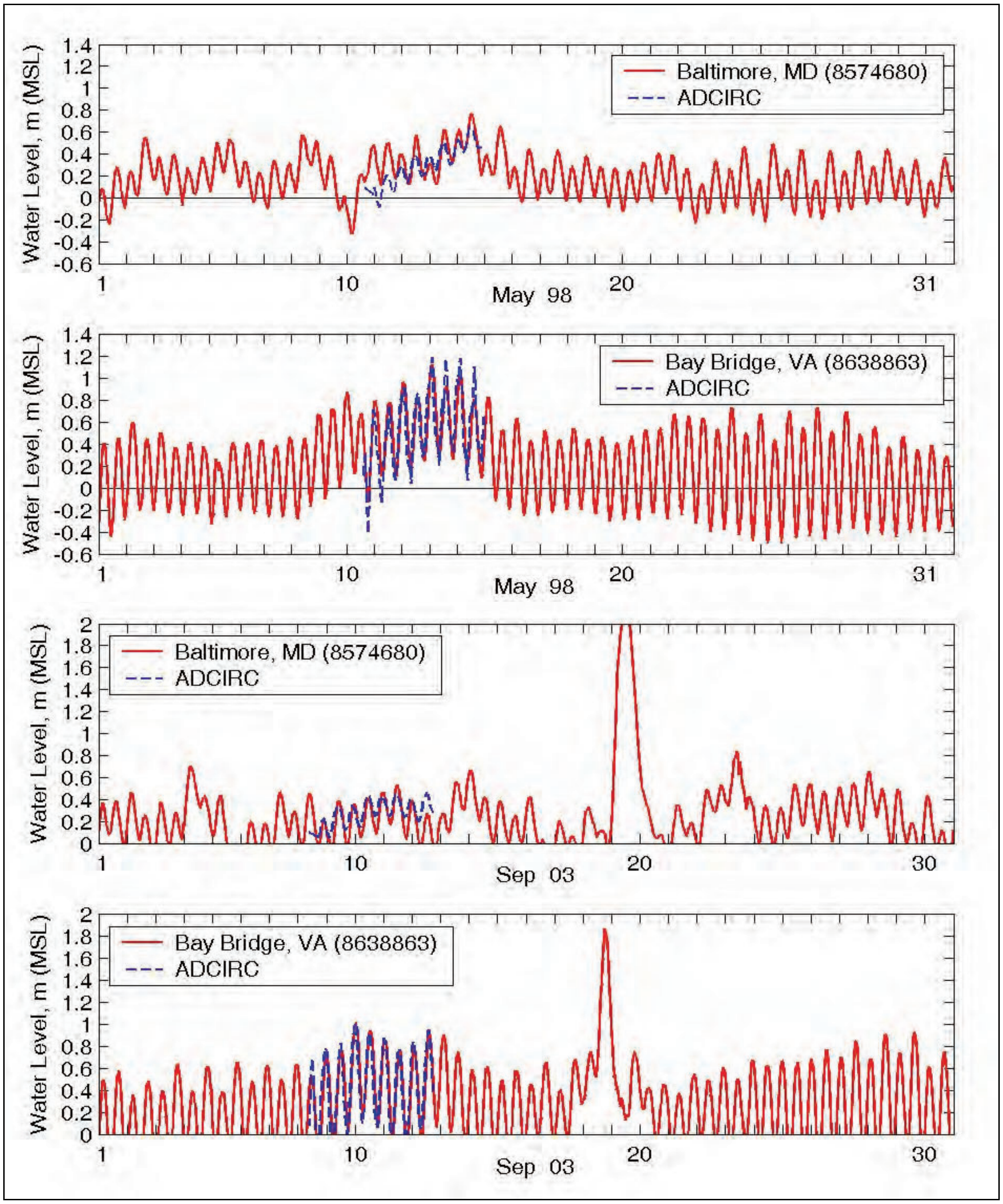


Figure 16. Measured and model water levels at sta 8574680 and 8638863 for May 1998 and September 2003

Extraction of Maximum Water Level and Water Level Time Series at Poplar, Barren, and James Islands

The validated model was applied to the suite of storms presented in Chapter 2 (Tables 1 and 2). Maximum water levels were extracted at 24 locations around Barren Island, James Island, and Poplar Island for use in the wave modeling and life-cycle analysis tasks (Figures 17-19 and Tables 10-12). Maximum water levels for tropical storms are shown in Tables 13-15. For instance, water levels predicted at Poplar Island for 52 hurricanes ranged between 0.33 and 2.44 m (1.1 and 8.0 ft). For a given storm, water levels vary from one side of Poplar Island to the other side by as much as 0.43 m (1.4 ft).

Maximum water levels for extratropical storms are shown in Tables 16-18. For instance, water levels at Poplar Island for these northeasters ranged between 0.19 and 0.98 m (0.6 and 3.2 ft). For a given storm, water levels vary from one side of Poplar Island to the other side by as much as 0.05 m (0.2 ft).

Time series of water level were extracted at a location for wave estimates around each island. The coordinates of these points are (38.33°N, 76.28°W) for Barren Island, (38.52°N, 76.37°W) for James Island, and (38.77°N, 76.4°W) for Poplar Island. Details of wave modeling are presented in Chapter 4.

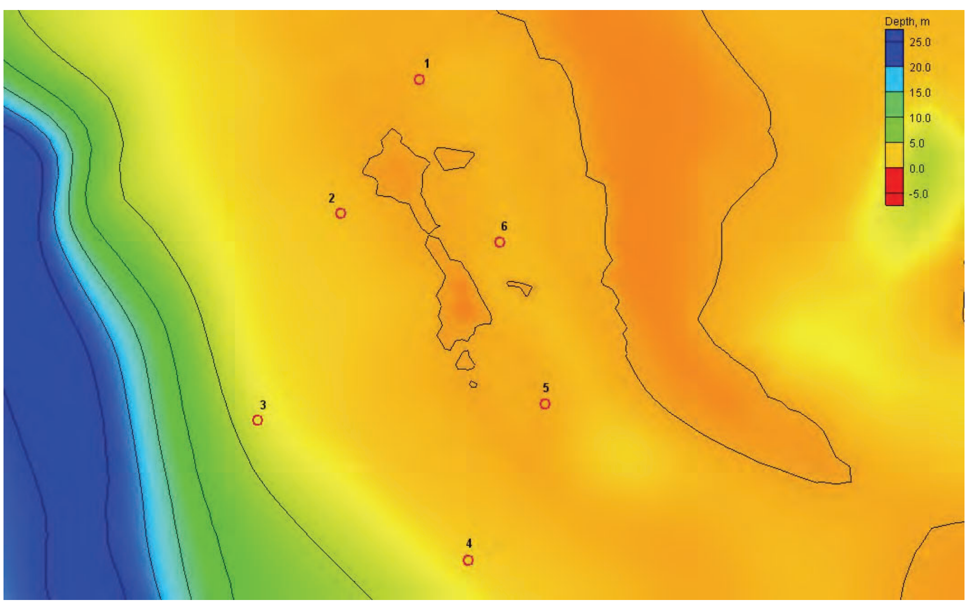


Figure 17. Six Barren Island save station locations for saving simulated water levels (red circles)

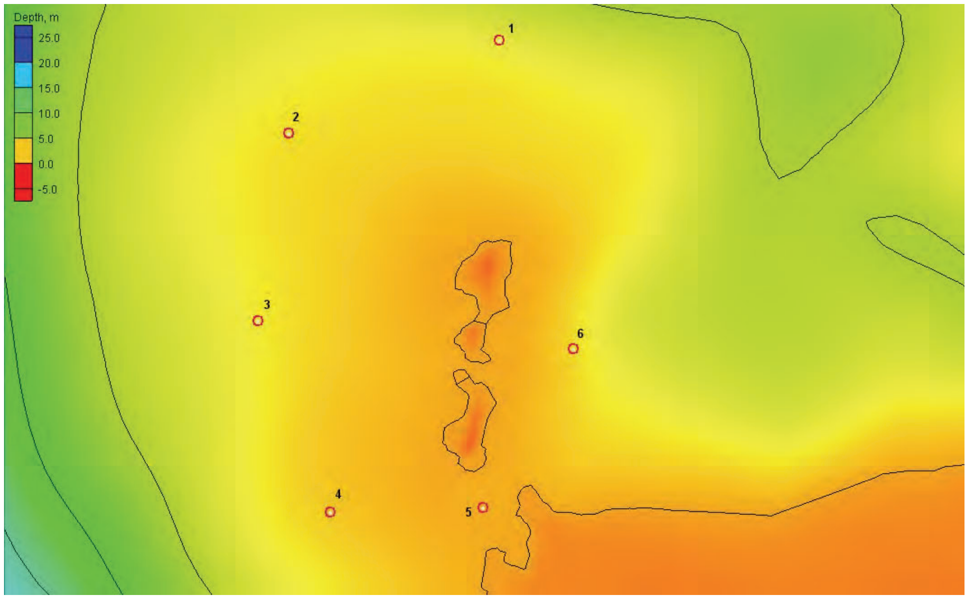


Figure 18. Six James Island save station locations for saving simulated water levels

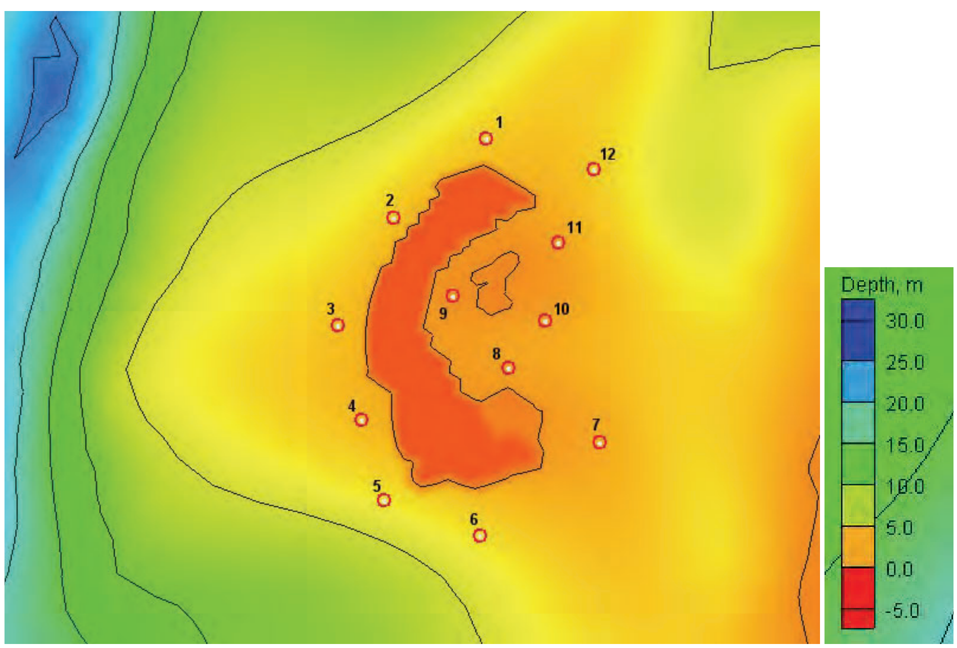


Figure 19. Twelve Poplar Island save station locations for saving simulated water levels

Table 10 **Barren Island Water Level Save Station Location Coordinates and Depths**		
Station No.	**Coordinates**	**Depth, m (ft), msl**
1	38^o 20' 29" N, 76^o 15' 10" W	0.30 (0.98)
2	38^o 19' 39" N, 76^o 15' 40" W	0.82 (2.69)
3	38^o 18' 20" N, 76^o 16' 12" W	3.15 (10.33)
4	38^o 17' 27" N, 76^o 14' 52" W	1.37 (4.49)
5	38^o 18' 26" N, 76^o 14' 23" W	0.23 (0.75)
6	38^o 19' 28" N, 76^o 14' 40" W	0.57 (1.87)

Table 11 **James Island Water Level Save Station Location Coordinates and Depths**		
Station No.	**Coordinates**	**Depth, m (ft), msl**
1	38^o 32' 31" N, 76^o 20' 13" W	2.75 (9.02)
2	38^o 32' 01" N, 76^o 21' 22" W	1.80 (5.91)
3	38^o 30' 59" N, 76^o 21' 32" W	2.15 (7.05)
4	38^o 29' 56" N, 76^o 21' 08" W	1.18 (3.87)
5	38^o 29' 58" N, 76^o 20' 18" W	0.25 (0.82)
6	38^o 30' 50" N, 76^o 19' 49" W	1.64 (5.38)

Table 12 **Poplar Island Water Level Save Station Location Coordinates and Depths**		
Station No.	**Coordinates**	**Depth, m (ft), msl**
1	38^o 46' 46" N, 76^o 22' 28" W	2.17 (7.12)
2	38^o 46' 27" N, 76^o 23' 02" W	1.99 (6.53)
3	38^o 45' 48" N, 76^o 23' 23" W	1.75 (5.74)
4	38^o 45' 13" N, 76^o 23' 14" W	1.58 (5.18)
5	38^o 44' 43" N, 76^o 23' 06" W	3.30 (10.83)
6	38^o 44' 30" N, 76^o 22' 31" W	2.97 (9.74)
7	38^o 45' 04" N, 76^o 21' 46" W	0.97 (3.18)
8	38^o 45' 32" N, 76^o 22' 20" W	0.73 (2.40)
9	38^o 45' 50" N, 76^o 22' 42" W	0.36 (1.18)
10	38^o 45' 49" N, 76^o 22' 07" W	0.76 (2.49)
11	38^o 46' 18" N, 76^o 22' 02" W	0.74 (2.43)
12	38^o 46' 45" N, 76^o 21' 49" W	2.18 (7.15)

Table 13
Peak Water Levels at Barren Island for 52 Tropical Storms for ADCIRC Save Station Locations, m (ft), msl

Storm	sta 1	sta 2	sta 3	sta 4	sta 5	sta 6
1	0.445 (1.46)	0.491 (1.61)	0.514 (1.69)	0.506 (1.66)	0.471 (1.55)	0.456 (1.50)
2	0.910 (2.99)	0.890 (2.92)	0.875 (2.87)	0.878 (2.88)	0.896 (2.94)	0.903 (2.96)
3	0.504 (1.65)	0.513 (1.68)	0.526 (1.73)	0.532 (1.75)	0.516 (1.69)	0.508 (1.67)
4	0.580 (1.90)	0.560 (1.84)	0.550 (1.80)	0.564 (1.85)	0.581 (1.91)	0.576 (1.89)
5	1.197 (3.93)	1.078 (3.54)	1.015 (3.33)	1.046 (3.43)	1.138 (3.73)	1.165 (3.82)
6	1.221 (4.01)	1.140 (3.74)	1.095 (3.59)	1.110 (3.64)	1.169 (3.84)	1.166 (3.83)
7	1.135 (3.72)	1.085 (3.56)	1.052 (3.45)	1.081 (3.55)	1.123 (3.68)	1.117 (3.66)
8	1.711 (5.61)	1.607 (5.27)	1.541 (5.06)	1.569 (5.15)	1.652 (5.42)	1.664 (5.46)
9	0.721 (2.37)	0.719 (2.36)	0.721 (2.37)	0.735 (2.41)	0.734 (2.41)	0.731 (2.40)
10	0.494 (1.62)	0.516 (1.69)	0.525 (1.72)	0.522 (1.71)	0.506 (1.66)	0.496 (1.63)
11	0.662 (2.17)	0.684 (2.24)	0.696 (2.28)	0.693 (2.27)	0.676 (2.22)	0.666 (2.19)
12	0.531 (1.74)	0.529 (1.74)	0.526 (1.73)	0.536 (1.76)	0.541 (1.77)	0.536 (1.76)
13	0.561 (1.84)	0.568 (1.86)	0.574 (1.88)	0.575 (1.89)	0.567 (1.86)	0.555 (1.82)
14	0.444 (1.46)	0.456 (1.50)	0.460 (1.51)	0.462 (1.52)	0.454 (1.49)	0.444 (1.46)
15	0.945 (3.10)	0.892 (2.93)	0.851 (2.79)	0.837 (2.75)	0.894 (2.93)	0.920 (3.02)
16	0.880 (2.89)	0.835 (2.74)	0.806 (2.64)	0.804 (2.64)	0.841 (2.76)	0.853 (2.80)
17	0.389 (1.28)	0.411 (1.35)	0.426 (1.40)	0.416 (1.36)	0.403 (1.32)	0.395 (1.30)
18	0.484 (1.59)	0.512 (1.68)	0.527 (1.73)	0.523 (1.72)	0.498 (1.63)	0.486 (1.59)
19	0.555 (1.82)	0.556 (1.82)	0.557 (1.83)	0.561 (1.84)	0.558 (1.83)	0.561 (1.84)
20	0.424 (1.39)	0.423 (1.39)	0.422 (1.38)	0.430 (1.41)	0.431 (1.41)	0.426 (1.40)
21	0.851 (2.79)	0.900 (2.95)	0.933 (3.06)	0.918 (3.01)	0.877 (2.88)	0.857 (2.81)
22	1.193 (3.91)	1.040 (3.41)	1.002 (3.29)	0.993 (3.26)	1.120 (3.67)	1.147 (3.76)
23	1.369 (4.49)	1.342 (4.40)	1.317 (4.32)	1.330 (4.36)	1.344 (4.41)	1.346 (4.42)
24	0.474 (1.56)	0.499 (1.64)	0.513 (1.68)	0.513 (1.68)	0.493 (1.62)	0.482 (1.58)
25	0.339 (1.11)	0.336 (1.10)	0.348 (1.14)	0.340 (1.12)	0.342 (1.12)	0.338 (1.11)
26	1.312 (4.30)	1.281 (4.20)	1.278 (4.19)	1.280 (4.20)	1.290 (4.23)	1.286 (4.22)
27	0.710 (2.33)	0.733 (2.40)	0.747 (2.45)	0.744 (2.44)	0.719 (2.36)	0.714 (2.34)
28	0.426 (1.40)	0.420 (1.38)	0.418 (1.37)	0.426 (1.40)	0.427 (1.40)	0.426 (1.40)
29	0.318 (1.04)	0.481 (1.58)	0.555 (1.82)	0.518 (1.70)	0.406 (1.33)	0.352 (1.15)
30	1.058 (3.47)	1.001 (3.28)	0.960 (3.15)	0.978 (3.21)	1.035 (3.40)	1.042 (3.42)
31	0.630 (2.07)	0.626 (2.05)	0.627 (2.06)	0.644 (2.11)	0.647 (2.12)	0.639 (2.10)
32	0.353 (1.16)	0.359 (1.18)	0.364 (1.19)	0.373 (1.22)	0.368 (1.21)	0.359 (1.18)
33	0.455 (1.49)	0.482 (1.58)	0.495 (1.62)	0.492 (1.61)	0.473 (1.55)	0.458 (1.50)
34	1.506 (4.94)	1.430 (4.69)	1.400 (4.59)	1.379 (4.52)	1.439 (4.72)	1.438 (4.72)
35	1.263 (4.14)	1.224 (4.02)	1.212 (3.98)	1.238 (4.06)	1.288 (4.23)	1.278 (4.19)
36	0.676 (2.22)	0.631 (2.07)	0.609 (2.00)	0.626 (2.05)	0.662 (2.17)	0.656 (2.15)
37	0.750 (2.46)	0.774 (2.54)	0.790 (2.59)	0.784 (2.57)	0.764 (2.51)	0.755 (2.48)
38	0.692 (2.27)	0.672 (2.20)	0.660 (2.17)	0.670 (2.20)	0.690 (2.26)	0.693 (2.27)
39	0.741 (2.43)	0.746 (2.45)	0.751 (2.46)	0.765 (2.51)	0.758 (2.49)	0.757 (2.48)
40	0.403 (1.32)	0.402 (1.32)	0.399 (1.31)	0.402 (1.32)	0.404 (1.33)	0.403 (1.32)
41	0.795 (2.61)	0.785 (2.58)	0.778 (2.55)	0.784 (2.57)	0.789 (2.59)	0.799 (2.62)
42	0.369 (1.21)	0.378 (1.24)	0.389 (1.28)	0.393 (1.29)	0.373 (1.22)	0.367 (1.20)
43	0.333 (1.09)	0.335 (1.10)	0.336 (1.10)	0.344 (1.13)	0.342 (1.12)	0.336 (1.10)
44	0.813 (2.67)	0.788 (2.59)	0.800 (2.62)	0.812 (2.66)	0.824 (2.70)	0.820 (2.69)
45	0.450 (1.48)	0.500 (1.64)	0.525 (1.72)	0.515 (1.69)	0.474 (1.56)	0.452 (1.48)
46	0.394 (1.29)	0.398 (1.31)	0.399 (1.31)	0.405 (1.33)	0.403 (1.32)	0.398 (1.31)
47	1.036 (3.40)	1.014 (3.33)	0.998 (3.27)	1.004 (3.29)	1.029 (3.38)	1.030 (3.38)
48	0.993 (3.26)	0.978 (3.21)	0.968 (3.18)	0.968 (3.18)	0.980 (3.22)	0.982 (3.22)
49	0.716 (2.35)	0.740 (2.43)	0.756 (2.48)	0.753 (2.47)	0.731 (2.40)	0.720 (2.36)
50	0.461 (1.51)	0.449 (1.47)	0.443 (1.45)	0.453 (1.49)	0.460 (1.51)	0.458 (1.50)
51	1.082 (3.55)	1.078 (3.54)	1.078 (3.54)	1.096 (3.60)	1.102 (3.62)	1.095 (3.59)
52	1.672 (5.49)	1.627 (5.34)	1.612 (5.29)	1.611 (5.29)	1.635 (5.36)	1.641 (5.38)

Table 14
Peak Water Levels at James Island for 52 Tropical Storms for ADCIRC Save Station Locations, m (ft), msl

Storm	sta 1	sta 2	sta 3	sta 4	sta 5	sta 6
1	0.511 (1.68)	0.506 (1.66)	0.501 (1.64)	0.492 (1.61)	0.488 (1.60)	0.528 (1.73)
2	0.886 (2.91)	0.883 (2.90)	0.884 (2.90)	0.885 (2.90)	0.883 (2.90)	0.885 (2.90)
3	0.485 (1.59)	0.494 (1.62)	0.494 (1.62)	0.487 (1.60)	0.480 (1.57)	0.489 (1.60)
4	0.515 (1.69)	0.513 (1.68)	0.514 (1.69)	0.514 (1.69)	0.507 (1.66)	0.506 (1.66)
5	1.165 (3.82)	1.127 (3.70)	1.102 (3.62)	1.083 (3.55)	1.113 (3.65)	1.159 (3.80)
6	1.187 (3.89)	1.201 (3.94)	1.210 (3.97)	1.192 (3.91)	1.128 (3.70)	1.134 (3.72)
7	0.959 (3.15)	0.955 (3.13)	0.964 (3.16)	0.971 (3.19)	0.962 (3.16)	0.960 (3.15)
8	1.620 (5.31)	1.596 (5.24)	1.575 (5.17)	1.551 (5.09)	1.566 (5.14)	1.602 (5.26)
9	0.682 (2.24)	0.673 (2.21)	0.673 (2.21)	0.671 (2.20)	0.682 (2.24)	0.691 (2.27)
10	0.536 (1.76)	0.536 (1.76)	0.534 (1.75)	0.531 (1.74)	0.524 (1.72)	0.541 (1.77)
11	0.690 (2.26)	0.688 (2.26)	0.685 (2.25)	0.679 (2.23)	0.675 (2.21)	0.697 (2.29)
12	0.522 (1.71)	0.517 (1.70)	0.515 (1.69)	0.531 (1.74)	0.517 (1.70)	0.523 (1.72)
13	0.603 (1.98)	0.606 (1.99)	0.602 (1.98)	0.596 (1.96)	0.588 (1.93)	0.600 (1.97)
14	0.474 (1.56)	0.473 (1.55)	0.470 (1.54)	0.467 (1.53)	0.462 (1.52)	0.471 (1.55)
15	1.187 (3.89)	1.160 (3.81)	1.131 (3.71)	1.105 (3.63)	1.127 (3.70)	1.170 (3.84)
16	1.013 (3.32)	1.005 (3.30)	0.992 (3.25)	0.978 (3.21)	0.975 (3.20)	0.992 (3.25)
17	0.452 (1.48)	0.450 (1.48)	0.443 (1.45)	0.437 (1.43)	0.432 (1.42)	0.473 (1.55)
18	0.534 (1.75)	0.538 (1.77)	0.539 (1.77)	0.536 (1.76)	0.523 (1.72)	0.538 (1.77)
19	0.568 (1.86)	0.562 (1.84)	0.558 (1.83)	0.556 (1.82)	0.562 (1.84)	0.575 (1.89)
20	0.436 (1.43)	0.429 (1.41)	0.421 (1.38)	0.417 (1.37)	0.419 (1.37)	0.435 (1.43)
21	0.922 (3.02)	0.930 (3.05)	0.931 (3.05)	0.924 (3.03)	0.902 (2.96)	0.928 (3.04)
22	1.254 (4.11)	1.220 (4.00)	1.199 (3.93)	1.179 (3.87)	1.182 (3.88)	1.223 (4.01)
23	1.168 (3.83)	1.159 (3.80)	1.164 (3.82)	1.168 (3.83)	1.167 (3.83)	1.169 (3.84)
24	0.509 (1.67)	0.507 (1.66)	0.507 (1.66)	0.504 (1.65)	0.501 (1.64)	0.519 (1.70)
25	0.364 (1.19)	0.358 (1.17)	0.352 (1.15)	0.345 (1.13)	0.342 (1.12)	0.403 (1.32)
26	1.205 (3.95)	1.200 (3.94)	1.202 (3.94)	1.200 (3.94)	1.141 (3.74)	1.147 (3.76)
27	0.736 (2.41)	0.730 (2.40)	0.726 (2.38)	0.722 (2.37)	0.721 (2.37)	0.748 (2.45)
28	0.422 (1.38)	0.426 (1.40)	0.425 (1.39)	0.419 (1.37)	0.414 (1.36)	0.420 (1.38)
29	0.509 (1.67)	0.516 (1.69)	0.518 (1.70)	0.512 (1.68)	0.468 (1.54)	0.557 (1.83)
30	1.051 (3.45)	1.042 (3.42)	1.035 (3.40)	1.027 (3.37)	1.035 (3.40)	1.049 (3.44)
31	0.605 (1.98)	0.599 (1.97)	0.596 (1.96)	0.595 (1.95)	0.602 (1.98)	0.606 (1.99)
32	0.350 (1.15)	0.346 (1.14)	0.341 (1.12)	0.337 (1.11)	0.342 (1.12)	0.352 (1.15)
33	0.497 (1.63)	0.504 (1.65)	0.506 (1.66)	0.502 (1.65)	0.489 (1.60)	0.497 (1.63)
34	1.698 (5.57)	1.685 (5.53)	1.663 (5.46)	1.640 (5.38)	1.648 (5.41)	1.676 (5.50)
35	1.095 (3.59)	1.085 (3.56)	1.094 (3.59)	1.110 (3.64)	1.119 (3.67)	1.113 (3.65)
36	0.639 (2.10)	0.639 (2.10)	0.636 (2.09)	0.628 (2.06)	0.621 (2.04)	0.620 (2.03)
37	0.774 (2.54)	0.776 (2.55)	0.776 (2.55)	0.774 (2.54)	0.766 (2.51)	0.780 (2.56)
38	0.699 (2.29)	0.694 (2.28)	0.687 (2.25)	0.678 (2.22)	0.678 (2.22)	0.704 (2.31)
39	0.656 (2.15)	0.667 (2.19)	0.667 (2.19)	0.663 (2.18)	0.664 (2.18)	0.665 (2.18)
40	0.433 (1.42)	0.427 (1.40)	0.422 (1.38)	0.419 (1.37)	0.424 (1.39)	0.435 (1.43)
41	0.795 (2.61)	0.787 (2.58)	0.784 (2.57)	0.783 (2.57)	0.795 (2.61)	0.808 (2.65)
42	0.396 (1.30)	0.391 (1.28)	0.392 (1.29)	0.390 (1.28)	0.386 (1.27)	0.398 (1.31)
43	0.337 (1.11)	0.333 (1.09)	0.328 (1.08)	0.342 (1.12)	0.328 (1.08)	0.336 (1.10)
44	0.726 (2.38)	0.718 (2.36)	0.713 (2.34)	0.711 (2.33)	0.712 (2.34)	0.721 (2.37)
45	0.527 (1.73)	0.527 (1.73)	0.530 (1.74)	0.527 (1.73)	0.506 (1.66)	0.533 (1.75)
46	0.400 (1.31)	0.395 (1.30)	0.395 (1.30)	0.397 (1.30)	0.397 (1.30)	0.402 (1.32)
47	0.944 (3.10)	0.955 (3.13)	0.967 (3.17)	0.974 (3.20)	0.967 (3.17)	0.953 (3.13)
48	1.029 (3.38)	1.029 (3.38)	1.026 (3.37)	1.019 (3.34)	1.014 (3.33)	1.014 (3.33)
49	0.753 (2.47)	0.757 (2.48)	0.755 (2.48)	0.753 (2.47)	0.742 (2.43)	0.751 (2.46)
50	0.464 (1.52)	0.456 (1.50)	0.448 (1.47)	0.443 (1.45)	0.450 (1.48)	0.463 (1.52)
51	1.036 (3.40)	1.040 (3.41)	1.042 (3.42)	1.041 (3.42)	0.992 (3.25)	1.070 (3.51)
52	1.684 (5.52)	1.690 (5.54)	1.687 (5.53)	1.669 (5.48)	1.635 (5.36)	1.642 (5.39)

Table 15 Peak Water Levels at Poplar Island for 52 Tropical Storms for ADCIRC Save Station Locations, m (ft), msl						
Storm	**sta 1**	**sta 2**	**sta 3**	**sta 4**	**sta 5**	**sta 6**
1	0.450 (1.48)	0.444 (1.46)	0.439 (1.44)	0.439 (1.44)	0.443 (1.45)	0.439 (1.44)
2	0.918 (3.01)	0.916 (3.01)	0.914 (3.00)	0.916 (3.01)	0.916 (3.01)	0.918 (3.01)
3	0.516 (1.69)	0.520 (1.71)	0.523 (1.72)	0.524 (1.72)	0.528 (1.73)	0.529 (1.74)
4	0.525 (1.72)	0.526 (1.73)	0.526 (1.73)	0.530 (1.74)	0.529 (1.74)	0.529 (1.74)
5	1.268 (4.16)	1.259 (4.13)	1.253 (4.11)	1.271 (4.17)	1.280 (4.20)	1.311 (4.30)
6	1.422 (4.67)	1.417 (4.65)	1.399 (4.59)	1.471 (4.83)	1.516 (4.97)	1.563 (5.13)
7	0.949 (3.11)	0.948 (3.11)	0.950 (3.12)	0.960 (3.15)	0.955 (3.13)	0.960 (3.15)
8	2.028 (6.65)	2.027 (6.65)	2.000 (6.56)	2.186 (7.17)	2.342 (7.68)	2.398 (7.87)
9	0.687 (2.25)	0.697 (2.29)	0.708 (2.32)	0.708 (2.32)	0.719 (2.36)	0.722 (2.37)
10	0.525 (1.72)	0.525 (1.72)	0.526 (1.73)	0.528 (1.73)	0.529 (1.74)	0.527 (1.73)
11	0.660 (2.17)	0.658 (2.16)	0.658 (2.16)	0.659 (2.16)	0.660 (2.17)	0.658 (2.16)
12	0.523 (1.72)	0.523 (1.72)	0.523 (1.72)	0.522 (1.71)	0.522 (1.71)	0.519 (1.70)
13	0.612 (2.01)	0.612 (2.01)	0.615 (2.02)	0.614 (2.01)	0.616 (2.02)	0.617 (2.02)
14	0.490 (1.61)	0.493 (1.62)	0.494 (1.62)	0.494 (1.62)	0.495 (1.62)	0.495 (1.62)
15	1.434 (4.70)	1.436 (4.71)	1.436 (4.71)	1.445 (4.74)	1.451 (4.76)	1.580 (5.18)
16	1.191 (3.91)	1.169 (3.84)	1.155 (3.79)	1.209 (3.97)	1.262 (4.14)	1.331 (4.37)
17	0.377 (1.24)	0.368 (1.21)	0.367 (1.20)	0.371 (1.22)	0.376 (1.23)	0.374 (1.23)
18	0.529 (1.74)	0.528 (1.73)	0.530 (1.74)	0.531 (1.74)	0.532 (1.75)	0.529 (1.74)
19	0.554 (1.82)	0.552 (1.81)	0.551 (1.81)	0.552 (1.81)	0.553 (1.81)	0.553 (1.81)
20	0.434 (1.42)	0.435 (1.43)	0.435 (1.43)	0.435 (1.43)	0.435 (1.43)	0.434 (1.42)
21	0.886 (2.91)	0.878 (2.88)	0.876 (2.87)	0.879 (2.88)	0.883 (2.90)	0.877 (2.88)
22	1.432 (4.70)	1.408 (4.62)	1.390 (4.56)	1.469 (4.82)	1.527 (5.01)	1.602 (5.26)
23	1.165 (3.82)	1.163 (3.82)	1.163 (3.82)	1.171 (3.84)	1.166 (3.83)	1.168 (3.83)
24	0.499 (1.64)	0.495 (1.62)	0.492 (1.61)	0.491 (1.61)	0.494 (1.62)	0.492 (1.61)
25	0.376 (1.23)	0.375 (1.23)	0.374 (1.23)	0.374 (1.23)	0.373 (1.22)	0.373 (1.22)
26	1.424 (4.67)	1.414 (4.64)	1.396 (4.58)	1.460 (4.79)	1.499 (4.92)	1.547 (5.08)
27	0.693 (2.27)	0.685 (2.25)	0.677 (2.22)	0.675 (2.21)	0.681 (2.23)	0.680 (2.23)
28	0.438 (1.44)	0.441 (1.45)	0.442 (1.45)	0.443 (1.45)	0.445 (1.46)	0.446 (1.46)
29	0.409 (1.34)	0.387 (1.27)	0.372 (1.22)	0.374 (1.23)	0.385 (1.26)	0.375 (1.23)
30	1.057 (3.47)	1.061 (3.48)	1.061 (3.48)	0.065 (0.21)	1.066 (3.50)	1.068 (3.50)
31	0.616 (2.02)	0.615 (2.02)	0.614 (2.01)	0.614 (2.01)	0.613 (2.01)	0.614 (2.01)
32	0.334 (1.10)	0.336 (1.10)	0.337 (1.11)	0.338 (1.11)	0.338 (1.11)	0.338 (1.11)
33	0.507 (1.66)	0.508 (1.67)	0.511 (1.68)	0.511 (1.68)	0.512 (1.68)	0.510 (1.67)
34	2.048 (6.72)	2.014 (6.61)	2.013 (6.60)	2.213 (7.26)	2.382 (7.81)	2.444 (8.02)
35	1.065 (3.49)	1.059 (3.47)	1.052 (3.45)	1.052 (3.45)	1.046 (3.43)	1.052 (3.45)
36	0.804 (2.64)	0.792 (2.60)	0.785 (2.58)	0.824 (2.70)	0.837 (2.75)	0.858 (2.81)
37	0.760 (2.49)	0.755 (2.48)	0.752 (2.47)	0.752 (2.47)	0.755 (2.48)	0.753 (2.47)
38	0.696 (2.28)	0.696 (2.28)	0.696 (2.28)	0.697 (2.29)	0.695 (2.28)	0.696 (2.28)
39	0.656 (2.15)	0.659 (2.16)	0.667 (2.19)	0.666 (2.19)	0.670 (2.20)	0.665 (2.18)
40	0.450 (1.48)	0.448 (1.47)	0.446 (1.46)	0.446 (1.46)	0.446 (1.46)	0.446 (1.46)
41	0.759 (2.49)	0.757 (2.48)	0.760 (2.49)	0.762 (2.50)	0.764 (2.51)	0.763 (2.50)
42	0.378 (1.24)	0.381 (1.25)	0.383 (1.26)	0.385 (1.26)	0.385 (1.26)	0.386 (1.27)
43	0.336 (1.10)	0.337 (1.11)	0.336 (1.10)	0.335 (1.10)	0.336 (1.10)	0.335 (1.10)
44	0.700 (2.30)	0.706 (2.32)	0.709 (2.33)	0.712 (2.34)	0.709 (2.33)	0.708 (2.32)
45	0.513 (1.68)	0.511 (1.68)	0.512 (1.68)	0.514 (1.69)	0.517 (1.70)	0.514 (1.69)
46	0.387 (1.27)	0.387 (1.27)	0.388 (1.27)	0.389 (1.28)	0.390 (1.28)	0.390 (1.28)
47	0.873 (2.86)	0.874 (2.87)	0.887 (2.91)	0.891 (2.92)	0.891 (2.92)	0.891 (2.92)
48	1.109 (3.64)	1.108 (3.64)	1.105 (3.63)	1.113 (3.65)	1.117 (3.66)	1.124 (3.69)
49	0.754 (2.47)	0.755 (2.48)	0.759 (2.49)	0.759 (2.49)	0.760 (2.49)	0.758 (2.49)
50	0.474 (1.56)	0.475 (1.56)	0.475 (1.56)	0.476 (1.56)	0.476 (1.56)	0.475 (1.56)
51	0.989 (3.24)	0.981 (3.22)	0.990 (3.25)	0.996 (3.27)	1.000 (3.28)	0.984 (3.23)
52	1.927 (6.32)	1.918 (6.29)	1.895 (6.22)	1.954 (6.41)	1.994 (6.54)	2.046 (6.71)

(Continued)

Table 15 (Concluded)

Storm	sta 7	sta 8	sta 9	sta 10	sta 11	sta 12
1	0.429 (1.41)	0.475 (1.56)	0.476 (1.56)	0.458 (1.50)	0.457 (1.50)	0.454 (1.49)
2	0.916 (3.01)	0.915 (3.00)	0.916 (3.01)	0.919 (3.02)	0.921 (3.02)	0.922 (3.02)
3	0.519 (1.70)	0.531 (1.74)	0.545 (1.79)	0.524 (1.72)	0.522 (1.71)	0.519 (1.70)
4	0.523 (1.72)	0.519 (1.70)	0.548 (1.80)	0.521 (1.71)	0.523 (1.72)	0.525 (1.72)
5	1.265 (4.15)	1.239 (4.06)	1.238 (4.06)	1.258 (4.13)	1.266 (4.15)	1.276 (4.19)
6	1.433 (4.70)	1.442 (4.73)	1.488 (4.88)	1.450 (4.76)	1.438 (4.72)	1.429 (4.69)
7	0.962 (3.16)	0.952 (3.12)	0.941 (3.09)	0.962 (3.16)	0.967 (3.17)	0.969 (3.18)
8	2.169 (7.12)	2.103 (6.90)	2.198 (7.21)	2.138 (7.01)	2.112 (6.93)	2.046 (6.71)
9	0.701 (2.33)	0.767 (2.52)	0.827 (2.71)	0.737 (2.42)	0.728 (2.39)	0.701 (2.30)
10	0.519 (1.70)	0.523 (1.72)	0.529 (1.74)	0.520 (1.71)	0.522 (1.71)	0.524 (1.72)
11	0.655 (2.15)	0.672 (2.20)	0.676 (2.22)	0.667 (2.19)	0.667 (2.19)	0.665 (2.18)
12	0.510 (1.67)	0.503 (1.65)	0.499 (1.64)	0.506 (1.66)	0.513 (1.68)	0.519 (1.70)
13	0.615 (2.02)	0.625 (2.05)	0.631 (2.07)	0.620 (2.03)	0.618 (2.03)	0.615 (2.02)
14	0.487 (1.60)	0.490 (1.61)	0.497 (1.63)	0.488 (1.60)	0.487 (1.60)	0.487 (1.60)
15	1.420 (4.66)	1.405 (4.61)	1.405 (4.61)	1.419 (4.66)	1.429 (4.69)	1.438 (4.72)
16	1.184 (3.88)	1.177 (3.86)	1.210 (3.97)	1.190 (3.90)	1.187 (3.89)	1.191 (3.91)
17	0.359 (1.18)	0.463 (1.52)	0.483 (1.58)	0.423 (1.39)	0.413 (1.35)	0.394 (1.29)
18	0.520 (1.71)	0.538 (1.77)	0.546 (1.79)	0.531 (1.74)	0.531 (1.74)	0.530 (1.74)
19	0.552 (1.81)	0.556 (1.82)	0.551 (1.81)	0.552 (1.81)	0.554 (1.82)	0.556 (1.82)
20	0.426 (1.40)	0.417 (1.37)	0.413 (1.35)	0.421 (1.38)	0.426 (1.40)	0.431 (1.41)
21	0.870 (2.85)	0.913 (3.00)	0.929 (3.05)	9.898 (32.47)	0.893 (2.93)	0.887 (2.91)
22	1.428 (4.69)	1.404 (4.61)	1.459 (4.79)	1.434 (4.70)	1.426 (4.68)	1.438 (4.72)
23	1.179 (3.87)	1.169 (3.84)	1.163 (3.82)	1.178 (3.86)	1.182 (3.88)	1.183 (3.88)
24	0.488 (1.60)	0.506 (1.66)	0.504 (1.65)	0.500 (1.64)	0.501 (1.64)	0.501 (1.64)
25	0.367 (1.20)	0.379 (1.24)	0.404 (1.33)	0.365 (1.20)	0.370 (1.21)	0.374 (1.23)
26	1.430 (4.69)	1.432 (4.70)	1.480 (4.86)	1.439 (4.72)	1.421 (4.66)	1.407 (4.62)
27	0.679 (2.23)	0.707 (2.32)	0.705 (2.31)	0.698 (2.29)	0.696 (2.28)	0.695 (2.28)
28	0.439 (1.44)	0.446 (1.46)	0.454 (1.49)	0.442 (1.45)	0.441 (1.45)	0.438 (1.44)
29	0.363 (1.19)	0.503 (1.65)	0.529 (1.74)	0.454 (1.49)	0.442 (1.45)	0.421 (1.38)
30	1.056 (3.46)	1.053 (3.45)	1.054 (3.46)	1.053 (3.45)	1.052 (3.45)	1.054 (3.46)
31	0.619 (2.03)	0.616 (2.02)	0.613 (2.01)	0.618 (2.03)	0.618 (2.03)	0.619 (2.03)
32	0.336 (1.10)	0.341 (1.12)	0.345 (1.13)	0.338 (1.11)	0.335 (1.10)	0.334 (1.10)
33	0.502 (1.65)	0.523 (1.72)	0.534 (1.75)	0.514 (1.69)	0.512 (1.68)	0.509 (1.67)
34	2.177 (7.14)	2.123 (6.97)	2.251 (7.39)	2.156 (7.07)	2.118 (6.95)	2.059 (6.76)
35	1.039 (3.41)	1.031 (3.38)	1.038 (3.41)	1.047 (3.44)	1.057 (3.47)	1.068 (3.50)
36	0.781 (2.56)	0.752 (2.47)	0.774 (2.54)	0.780 (2.56)	0.787 (2.58)	0.801 (2.63)
37	0.751 (2.46)	0.775 (2.54)	0.779 (2.56)	0.767 (2.52)	0.765 (2.51)	0.763 (2.50)
38	0.691 (2.27)	0.686 (2.25)	0.695 (2.28)	0.691 (2.27)	0.694 (2.28)	0.697 (2.29)
39	0.661 (2.17)	0.717 (2.35)	0.754 (2.47)	0.693 (2.27)	0.684 (2.24)	0.668 (2.19)
40	0.448 (1.47)	0.449 (1.47)	0.449 (1.47)	0.450 (1.48)	0.450 (1.48)	0.450 (1.48)
41	0.761 (2.50)	0.767 (2.52)	0.766 (2.51)	0.764 (2.51)	0.763 (2.50)	0.762 (2.50)
42	0.383 (1.26)	0.381 (1.25)	0.380 (1.25)	0.381 (1.25)	0.382 (1.25)	0.382 (1.25)
43	0.331 (1.09)	0.329 (1.08)	0.326 (1.07)	0.330 (1.08)	0.333 (1.09)	0.335 (1.10)
44	0.685 (2.25)	0.673 (2.21)	0.755 (2.48)	0.681 (2.23)	0.689 (2.26)	0.695 (2.28)
45	0.505 (1.66)	0.536 (1.76)	0.548 (1.80)	0.524 (1.72)	0.521 (1.71)	0.516 (1.69)
46	0.385 (1.26)	0.380 (1.25)	0.376 (1.23)	0.382 (1.25)	0.386 (1.27)	0.388 (1.27)
47	0.897 (2.94)	0.883 (2.90)	0.928 (3.04)	0.891 (2.92)	0.891 (2.92)	0.888 (2.91)
48	1.105 (3.63)	1.107 (3.63)	1.118 (3.67)	1.108 (3.64)	1.106 (3.63)	1.106 (3.63)
49	0.752 (2.47)	0.765 (2.51)	0.774 (2.54)	0.760 (2.49)	0.759 (2.49)	0.757 (2.48)
50	0.464 (1.52)	0.452 (1.48)	0.450 (1.48)	0.459 (1.51)	0.465 (1.53)	0.471 (1.55)
51	0.968 (3.18)	1.083 (3.55)	1.120 (3.67)	1.038 (3.41)	1.022 (3.35)	1.003 (3.29)
52	1.946 (6.38)	1.965 (6.45)	2.012 (6.60)	1.966 (6.45)	1.943 (6.37)	1.915 (6.28)

Table 16
Peak Water Levels at Barren Island for 43 Northeaster Storms for ADCIRC Save Station Locations, m (ft), msl

Storm	sta 1	sta 2	sta 3	sta 4	sta 5	sta 6
1	0.376 (1.23)	0.395 (1.30)	0.401 (1.32)	0.403 (1.32)	0.387 (1.27)	0.384 (1.26)
2	0.445 (1.46)	0.457 (1.50)	0.468 (1.54)	0.470 (1.54)	0.458 (1.50)	0.452 (1.48)
3	0.898 (2.95)	0.908 (2.98)	0.910 (2.99)	0.912 (2.99)	0.905 (2.97)	0.900 (2.95)
4	0.384 (1.26)	0.471 (1.55)	0.494 (1.62)	0.488 (1.60)	0.433 (1.42)	0.424 (1.39)
5	0.372 (1.22)	0.346 (1.14)	0.329 (1.08)	0.335 (1.10)	0.355 (1.16)	0.376 (1.23)
6	0.860 (2.82)	0.884 (2.90)	0.894 (2.93)	0.892 (2.93)	0.874 (2.87)	0.864 (2.83)
7	0.591 (1.94)	0.635 (2.08)	0.650 (2.13)	0.645 (2.12)	0.615 (2.02)	0.610 (2.00)
8	0.759 (2.49)	0.796 (2.61)	0.813 (2.67)	0.808 (2.65)	0.783 (2.57)	0.769 (2.52)
9	0.265 (0.87)	0.273 (0.90)	0.282 (0.93)	0.278 (0.91)	0.239 (0.78)	0.249 (0.82)
10	0.453 (1.49)	0.486 (1.59)	0.499 (1.64)	0.499 (1.64)	0.479 (1.57)	0.467 (1.53)
11	0.711 (2.33)	0.740 (2.43)	0.753 (2.47)	0.748 (2.45)	0.729 (2.39)	0.718 (2.36)
12	0.688 (2.26)	0.697 (2.29)	0.704 (2.31)	0.708 (2.32)	0.699 (2.29)	0.691 (2.27)
13	0.891 (2.92)	0.854 (2.80)	0.833 (2.73)	0.850 (2.79)	0.879 (2.88)	0.881 (2.89)
14	0.343 (1.13)	0.384 (1.26)	0.393 (1.29)	0.392 (1.29)	0.366 (1.20)	0.361 (1.18)
15	0.608 (1.99)	0.633 (2.08)	0.642 (2.11)	0.639 (2.10)	0.622 (2.04)	0.613 (2.01)
16	0.594 (1.95)	0.621 (2.04)	0.634 (2.08)	0.636 (2.09)	0.615 (2.02)	0.603 (1.98)
17	0.518 (1.70)	0.541 (1.77)	0.550 (1.80)	0.546 (1.79)	0.530 (1.74)	0.521 (1.71)
18	0.586 (1.92)	0.593 (1.95)	0.593 (1.95)	0.594 (1.95)	0.590 (1.94)	0.586 (1.92)
19	0.529 (1.74)	0.546 (1.79)	0.553 (1.81)	0.551 (1.81)	0.538 (1.77)	0.533 (1.75)
20	0.861 (2.82)	0.832 (2.73)	0.815 (2.67)	0.823 (2.70)	0.843 (2.77)	0.860 (2.82)
21	0.560 (1.84)	0.592 (1.94)	0.605 (1.98)	0.597 (1.96)	0.579 (1.90)	0.569 (1.87)
22	0.461 (1.51)	0.512 (1.68)	0.528 (1.73)	0.526 (1.73)	0.486 (1.59)	0.483 (1.58)
23	0.255 (0.84)	0.321 (1.05)	0.335 (1.10)	0.337 (1.11)	0.310 (1.02)	0.291 (0.95)
24	0.892 (2.93)	0.907 (2.98)	0.915 (3.00)	0.914 (3.00)	0.896 (2.94)	0.889 (2.92)
25	0.892 (2.93)	0.902 (2.96)	0.906 (2.97)	0.910 (2.99)	0.903 (2.96)	0.892 (2.93)
26	0.299 (0.98)	0.322 (1.06)	0.324 (1.06)	0.327 (1.07)	0.317 (1.04)	0.311 (1.02)
27	0.512 (1.68)	0.546 (1.79)	0.559 (1.83)	0.559 (1.83)	0.537 (1.76)	0.524 (1.72)
28	0.552 (1.81)	0.589 (1.93)	0.602 (1.98)	0.603 (1.98)	0.577 (1.89)	0.568 (1.86)
29	0.465 (1.53)	0.476 (1.56)	0.482 (1.58)	0.484 (1.59)	0.468 (1.54)	0.470 (1.54)
30	0.585 (1.92)	0.594 (1.95)	0.594 (1.95)	0.594 (1.95)	0.588 (1.93)	0.587 (1.93)
31	0.407 (1.34)	0.426 (1.40)	0.436 (1.43)	0.447 (1.47)	0.431 (1.41)	0.418 (1.37)
32	0.314 (1.03)	0.374 (1.23)	0.390 (1.28)	0.386 (1.27)	0.354 (1.16)	0.338 (1.11)
33	0.749 (2.46)	0.733 (2.40)	0.725 (2.38)	0.735 (2.41)	0.749 (2.46)	0.751 (2.46)
34	0.432 (1.42)	0.453 (1.49)	0.457 (1.50)	0.459 (1.51)	0.447 (1.47)	0.438 (1.44)
35	0.564 (1.85)	0.560 (1.84)	0.562 (1.84)	0.566 (1.86)	0.560 (1.84)	0.567 (1.86)
36	0.554 (1.82)	0.556 (1.82)	0.567 (1.86)	0.568 (1.86)	0.552 (1.81)	0.549 (1.80)
37	0.420 (1.38)	0.425 (1.39)	0.428 (1.40)	0.432 (1.42)	0.423 (1.39)	0.424 (1.39)
38	0.690 (2.26)	0.698 (2.29)	0.700 (2.30)	0.705 (2.31)	0.698 (2.29)	0.694 (2.28)
39	0.460 (1.51)	0.487 (1.60)	0.498 (1.63)	0.497 (1.63)	0.472 (1.55)	0.469 (1.54)
40	0.662 (2.17)	0.658 (2.16)	0.653 (2.14)	0.660 (2.17)	0.662 (2.17)	0.659 (2.16)
41	0.588 (1.93)	0.593 (1.95)	0.595 (1.95)	0.599 (1.97)	0.597 (1.96)	0.593 (1.95)
42	0.606 (1.99)	0.698 (2.29)	0.702 (2.30)	0.705 (2.31)	0.698 (2.29)	0.691 (2.27)
43	0.490 (1.61)	0.506 (1.66)	0.514 (1.69)	0.513 (1.68)	0.499 (1.64)	0.494 (1.62)

Table 17
Peak Water Levels at James Island for 43 Northeaster Storms for ADCIRC Save Station Locations, m (ft), msl

Storm	sta 1	sta 2	sta 3	sta 4	sta 5	sta 6
1	0.423 (1.39)	0.420 (1.38)	0.417 (1.37)	0.414 (1.36)	0.418 (1.37)	0.431 (1.41)
2	0.469 (1.54)	0.466 (1.53)	0.466 (1.53)	0.463 (1.52)	0.459 (1.51)	0.479 (1.57)
3	0.919 (3.02)	0.918 (3.01)	0.915 (3.00)	0.911 (2.99)	0.909 (2.98)	0.919 (3.02)
4	0.485 (1.59)	0.479 (1.57)	0.476 (1.56)	0.474 (1.56)	0.482 (1.58)	0.510 (1.67)
5	0.367 (1.20)	0.353 (1.16)	0.344 (1.13)	0.339 (1.11)	0.367 (1.20)	0.382 (1.25)
6	0.887 (2.91)	0.891 (2.92)	0.891 (2.92)	0.888 (2.91)	0.878 (2.88)	0.887 (2.91)
7	0.646 (2.12)	0.640 (2.10)	0.638 (2.09)	0.633 (2.08)	0.637 (2.09)	0.664 (2.18)
8	0.798 (2.62)	0.798 (2.62)	0.797 (2.61)	0.794 (2.60)	0.784 (2.57)	0.805 (2.64)
9	0.278 (0.91)	0.264 (0.87)	0.259 (0.85)	0.256 (0.84)	0.284 (0.93)	0.303 (0.99)
10	0.514 (1.69)	0.507 (1.66)	0.505 (1.66)	0.501 (1.64)	0.498 (1.63)	0.526 (1.73)
11	0.752 (2.47)	0.750 (2.46)	0.747 (2.45)	0.742 (2.43)	0.735 (2.41)	0.756 (2.48)
12	0.710 (2.33)	0.707 (2.32)	0.703 (2.31)	0.697 (2.29)	0.697 (2.29)	0.714 (2.34)
13	0.885 (2.90)	0.871 (2.86)	0.864 (2.83)	0.857 (2.81)	0.860 (2.82)	0.876 (2.87)
14	0.409 (1.34)	0.401 (1.32)	0.396 (1.30)	0.393 (1.29)	0.396 (1.30)	0.419 (1.37)
15	0.660 (2.17)	0.659 (2.16)	0.657 (2.16)	0.654 (2.15)	0.647 (2.12)	0.664 (2.18)
16	0.636 (2.09)	0.630 (2.07)	0.629 (2.06)	0.627 (2.06)	0.627 (2.06)	0.649 (2.13)
17	0.565 (1.85)	0.565 (1.85)	0.562 (1.84)	0.557 (1.83)	0.551 (1.81)	0.569 (1.87)
18	0.622 (2.04)	0.618 (2.03)	0.614 (2.01)	0.608 (1.99)	0.609 (2.00)	0.624 (2.05)
19	0.565 (1.85)	0.562 (1.84)	0.558 (1.83)	0.554 (1.82)	0.549 (1.80)	0.571 (1.87)
20	0.865 (2.84)	0.849 (2.79)	0.841 (2.76)	0.837 (2.75)	0.867 (2.84)	0.883 (2.90)
21	0.627 (2.06)	0.619 (2.03)	0.612 (2.01)	0.605 (1.98)	0.605 (1.98)	0.641 (2.10)
22	0.524 (1.72)	0.512 (1.68)	0.511 (1.68)	0.510 (1.67)	0.514 (1.69)	0.541 (1.77)
23	0.325 (1.07)	0.315 (1.03)	0.306 (1.00)	0.300 (0.98)	0.315 (1.03)	0.336 (1.10)
24	0.934 (3.06)	0.937 (3.07)	0.934 (3.06)	0.932 (3.06)	0.923 (3.03)	0.929 (3.05)
25	0.916 (3.01)	0.913 (3.00)	0.912 (2.99)	0.909 (2.98)	0.906 (2.97)	0.912 (2.99)
26	0.355 (1.16)	0.351 (1.15)	0.346 (1.14)	0.341 (1.12)	0.348 (1.14)	0.362 (1.19)
27	0.567 (1.86)	0.564 (1.85)	0.562 (1.84)	0.558 (1.83)	0.557 (1.83)	0.576 (1.89)
28	0.606 (1.99)	0.603 (1.98)	0.601 (1.97)	0.597 (1.96)	0.599 (1.97)	0.622 (2.04)
29	0.493 (1.62)	0.486 (1.59)	0.483 (1.58)	0.481 (1.58)	0.489 (1.60)	0.503 (1.65)
30	0.627 (2.06)	0.621 (2.04)	0.613 (2.01)	0.606 (1.99)	0.606 (1.99)	0.630 (2.07)
31	0.447 (1.47)	0.442 (1.45)	0.438 (1.44)	0.434 (1.42)	0.439 (1.44)	0.453 (1.49)
32	0.393 (1.29)	0.389 (1.28)	0.386 (1.27)	0.383 (1.26)	0.382 (1.25)	0.405 (1.33)
33	0.739 (2.42)	0.729 (2.39)	0.723 (2.37)	0.719 (2.36)	0.737 (2.42)	0.749 (2.46)
34	0.486 (1.59)	0.480 (1.57)	0.477 (1.56)	0.474 (1.56)	0.473 (1.55)	0.492 (1.61)
35	0.568 (1.86)	0.568 (1.86)	0.566 (1.86)	0.561 (1.84)	0.562 (1.84)	0.574 (1.88)
36	0.594 (1.95)	0.588 (1.93)	0.580 (1.90)	0.575 (1.89)	0.578 (1.90)	0.596 (1.96)
37	0.414 (1.36)	0.408 (1.34)	0.404 (1.33)	0.401 (1.32)	0.411 (1.35)	0.423 (1.39)
38	0.719 (2.36)	0.711 (2.33)	0.705 (2.31)	0.704 (2.31)	0.706 (2.32)	0.725 (2.38)
39	0.521 (1.71)	0.511 (1.68)	0.509 (1.67)	0.505 (1.66)	0.505 (1.66)	0.534 (1.75)
40	0.676 (2.22)	0.673 (2.21)	0.669 (2.19)	0.664 (2.18)	0.665 (2.18)	0.671 (2.20)
41	0.615 (2.02)	0.607 (1.99)	0.604 (1.98)	0.603 (1.98)	0.606 (1.99)	0.616 (2.02)
42	0.716 (2.35)	0.710 (2.33)	0.709 (2.33)	0.707 (2.32)	0.707 (2.32)	0.725 (2.38)
43	0.531 (1.74)	0.526 (1.73)	0.522 (1.71)	0.516 (1.69)	0.516 (1.69)	0.540 (1.77)

Table 18
Peak Water Levels at Poplar Island for 43 Northeaster Storms for ADCIRC Save Station Locations, m (ft), msl

Storm	sta 1	sta 2	sta 3	sta 4	sta 5	sta 6
1	0.377 (1.24)	0.374 (1.23)	0.371 (1.22)	0.370 (1.21)	0.373 (1.22)	0.372 (1.22)
2	0.451 (1.48)	0.451 (1.48)	0.449 (1.47)	0.448 (1.47)	0.450 (1.48)	0.449 (1.47)
3	0.922 (3.02)	0.921 (3.02)	0.921 (3.02)	0.922 (3.02)	0.922 (3.02)	0.920 (3.02)
4	0.396 (1.30)	0.382 (1.25)	0.373 (1.22)	0.370 (1.21)	0.377 (1.24)	0.375 (1.23)
5	0.236 (0.77)	0.238 (0.78)	0.239 (0.78)	0.240 (0.79)	0.239 (0.78)	0.241 (0.79)
6	0.885 (2.90)	0.885 (2.90)	0.887 (2.91)	0.888 (2.91)	0.889 (2.92)	0.888 (2.91)
7	0.502 (1.65)	0.493 (1.62)	0.485 (1.59)	0.483 (1.58)	0.489 (1.60)	0.487 (1.60)
8	0.760 (2.49)	0.754 (2.47)	0.751 (2.46)	0.753 (2.47)	0.757 (2.48)	0.754 (2.47)
9	0.226 (0.74)	0.230 (0.75)	0.231 (0.76)	0.232 (0.76)	0.228 (0.75)	0.230 (0.75)
10	0.484 (1.59)	0.480 (1.57)	0.478 (1.57)	0.478 (1.57)	0.481 (1.58)	0.479 (1.57)
11	0.733 (2.40)	0.729 (2.39)	0.728 (2.39)	0.730 (2.40)	0.732 (2.40)	0.731 (2.40)
12	0.684 (2.24)	0.681 (2.23)	0.678 (2.22)	0.679 (2.23)	0.680 (2.23)	0.679 (2.23)
13	0.949 (3.11)	0.946 (3.10)	0.945 (3.10)	0.950 (3.12)	0.949 (3.11)	0.952 (3.12)
14	0.375 (1.23)	0.371 (1.22)	0.368 (1.21)	0.368 (1.21)	0.371 (1.22)	0.370 (1.21)
15	0.644 (2.11)	0.642 (2.11)	0.642 (2.11)	0.643 (2.11)	0.645 (2.12)	0.644 (2.11)
16	0.595 (1.95)	0.589 (1.93)	0.583 (1.91)	0.583 (1.91)	0.587 (1.93)	0.586 (1.92)
17	0.555 (1.82)	0.556 (1.82)	0.558 (1.83)	0.558 (1.83)	0.560 (1.84)	0.558 (1.83)
18	0.632 (2.07)	0.634 (2.08)	0.634 (2.08)	0.634 (2.08)	0.635 (2.08)	0.634 (2.08)
19	0.440 (1.44)	0.436 (1.43)	0.434 (1.42)	0.434 (1.42)	0.436 (1.43)	0.434 (1.42)
20	0.748 (2.45)	0.753 (2.47)	0.750 (2.46)	0.744 (2.44)	0.736 (2.41)	0.738 (2.42)
21	0.598 (1.96)	0.596 (1.96)	0.596 (1.96)	0.598 (1.96)	0.600 (1.97)	0.598 (1.96)
22	0.463 (1.52)	0.453 (1.49)	0.445 (1.46)	0.443 (1.45)	0.450 (1.48)	0.450 (1.48)
23	0.296 (0.97)	0.292 (0.96)	0.287 (0.94)	0.287 (0.94)	0.291 (0.95)	0.290 (0.95)
24	0.959 (3.15)	0.963 (3.16)	0.965 (3.17)	0.968 (3.18)	0.970 (3.18)	0.972 (3.19)
25	0.936 (3.07)	0.938 (3.08)	0.940 (3.08)	0.941 (3.09)	0.941 (3.09)	0.941 (3.09)
26	0.250 (0.82)	0.250 (0.82)	0.247 (0.81)	0.247 (0.81)	0.249 (0.82)	0.247 (0.81)
27	0.543 (1.78)	0.537 (1.76)	0.534 (1.75)	0.534 (1.75)	0.538 (1.77)	0.536 (1.76)
28	0.463 (1.52)	0.456 (1.50)	0.449 (1.47)	0.448 (1.47)	0.453 (1.49)	0.452 (1.48)
29	0.481 (1.58)	0.479 (1.57)	0.478 (1.57)	0.479 (1.57)	0.480 (1.57)	0.480 (1.57)
30	0.621 (2.04)	0.621 (2.04)	0.621 (2.04)	0.622 (2.04)	0.623 (2.04)	0.622 (2.04)
31	0.441 (1.45)	0.440 (1.44)	0.439 (1.44)	0.438 (1.44)	0.440 (1.44)	0.440 (1.44)
32	0.442 (1.45)	0.433 (1.42)	0.425 (1.39)	0.424 (1.39)	0.430 (1.41)	0.429 (1.41)
33	0.594 (1.95)	0.590 (1.94)	0.584 (1.92)	0.582 (1.91)	0.584 (1.92)	0.586 (1.92)
34	0.479 (1.57)	0.477 (1.56)	0.477 (1.56)	0.477 (1.56)	0.479 (1.57)	0.477 (1.56)
35	0.548 (1.80)	0.545 (1.79)	0.546 (1.79)	0.548 (1.80)	0.550 (1.80)	0.548 (1.80)
36	0.598 (1.96)	0.599 (1.97)	0.599 (1.97)	0.600 (1.97)	0.600 (1.97)	0.600 (1.97)
37	0.371 (1.22)	0.368 (1.21)	0.365 (1.20)	0.364 (1.19)	0.366 (1.20)	0.367 (1.20)
38	0.706 (2.32)	0.704 (2.31)	0.703 (2.31)	0.702 (2.30)	0.703 (2.31)	0.702 (2.30)
39	0.485 (1.59)	0.482 (1.58)	0.482 (1.58)	0.482 (1.58)	0.483 (1.58)	0.482 (1.58)
40	0.688 (2.26)	0.691 (2.27)	0.691 (2.27)	0.693 (2.27)	0.693 (2.27)	0.693 (2.27)
41	0.611 (2.00)	0.611 (2.00)	0.611 (2.00)	0.612 (2.01)	0.613 (2.01)	0.612 (2.01)
42	0.691 (2.27)	0.688 (2.26)	0.686 (2.25)	0.686 (2.25)	0.689 (2.26)	0.688 (2.26)
43	0.522 (1.71)	0.521 (1.71)	0.519 (1.70)	0.518 (1.70)	0.519 (1.70)	0.518 (1.70)

(Continued)

Table 18 (Concluded)						
Storm	**sta 7**	**sta 8**	**sta 9**	**sta 10**	**sta 11**	**sta 12**
1	0.370 (1.21)	0.379 (1.24)	0.377 (1.24)	0.375 (1.23)	0.376 (1.23)	0.376 (1.23)
2	0.440 (1.44)	0.460 (1.51)	0.465 (1.53)	0.451 (1.48)	0.450 (1.48)	0.448 (1.47)
3	0.918 (3.01)	0.927 (3.04)	0.931 (3.05)	0.923 (3.03)	0.923 (3.03)	0.922 (3.02)
4	0.365 (1.20)	0.419 (1.37)	0.411 (1.35)	0.401 (1.32)	0.402 (1.32)	0.399 (1.31)
5	0.243 (0.80)	0.232 (0.76)	0.214 (0.70)	0.234 (0.77)	0.237 (0.78)	0.240 (0.79)
6	0.883 (2.90)	0.894 (2.93)	0.901 (2.96)	0.890 (2.92)	0.888 (2.91)	0.886 (2.91)
7	0.481 (1.58)	0.511 (1.68)	0.512 (1.68)	0.501 (1.64)	0.502 (1.65)	0.502 (1.65)
8	0.749 (2.46)	0.773 (2.54)	0.778 (2.55)	0.765 (2.51)	0.763 (2.50)	0.761 (2.50)
9	0.220 (0.72)	0.206 (0.68)	0.186 (0.61)	0.211 (0.69)	0.218 (0.72)	0.229 (0.75)
10	0.470 (1.54)	0.484 (1.59)	0.487 (1.60)	0.479 (1.57)	0.481 (1.58)	0.483 (1.58)
11	0.727 (2.39)	0.744 (2.44)	0.749 (2.46)	0.738 (2.42)	0.735 (2.41)	0.733 (2.40)
12	0.674 (2.21)	0.687 (2.25)	0.694 (2.28)	0.683 (2.24)	0.682 (2.24)	0.683 (2.24)
13	0.938 (3.08)	0.927 (3.04)	0.928 (3.04)	0.937 (3.07)	0.943 (3.09)	0.950 (3.12)
14	0.361 (1.18)	0.373 (1.22)	0.368 (1.21)	0.369 (1.21)	0.373 (1.22)	0.374 (1.23)
15	0.640 (2.10)	0.656 (2.15)	0.663 (2.18)	0.651 (2.14)	0.650 (2.13)	0.647 (2.12)
16	0.583 (1.91)	0.603 (1.98)	0.602 (1.98)	0.596 (1.96)	0.595 (1.95)	0.595 (1.95)
17	0.550 (1.80)	0.561 (1.84)	0.566 (1.86)	0.556 (1.82)	0.555 (1.82)	0.555 (1.82)
18	0.628 (2.06)	0.630 (2.07)	0.631 (2.07)	0.628 (2.06)	0.629 (2.06)	0.630 (2.07)
19	0.429 (1.41)	0.446 (1.46)	0.447 (1.47)	0.439 (1.44)	0.441 (1.45)	0.442 (1.45)
20	0.745 (2.44)	0.734 (2.41)	0.714 (2.34)	0.738 (2.42)	0.742 (2.43)	0.750 (2.46)
21	0.589 (1.93)	0.604 (1.98)	0.609 (2.00)	0.598 (1.96)	0.598 (1.96)	0.598 (1.96)
22	0.447 (1.47)	0.476 (1.56)	0.472 (1.55)	0.466 (1.53)	0.467 (1.53)	0.465 (1.53)
23	0.281 (0.92)	0.288 (0.94)	0.277 (0.91)	0.286 (0.94)	0.293 (0.96)	0.295 (0.97)
24	0.966 (3.17)	0.972 (3.19)	0.980 (3.22)	0.969 (3.18)	0.966 (3.17)	0.962 (3.16)
25	0.936 (3.07)	0.937 (3.07)	0.939 (3.08)	0.936 (3.07)	0.936 (3.07)	0.937 (3.07)
26	0.239 (0.78)	0.238 (0.78)	0.230 (0.75)	0.239 (0.78)	0.244 (0.80)	0.248 (0.81)
27	0.530 (1.74)	0.546 (1.79)	0.546 (1.79)	0.540 (1.77)	0.542 (1.78)	0.543 (1.78)
28	0.447 (1.47)	0.467 (1.53)	0.464 (1.52)	0.460 (1.51)	0.462 (1.52)	0.464 (1.52)
29	0.482 (1.58)	0.487 (1.60)	0.483 (1.58)	0.484 (1.59)	0.483 (1.58)	0.483 (1.58)
30	0.618 (2.03)	0.623 (2.04)	0.626 (2.05)	0.621 (2.04)	0.621 (2.04)	0.622 (2.04)
31	0.434 (1.42)	0.431 (1.41)	0.426 (1.40)	0.432 (1.42)	0.435 (1.43)	0.439 (1.44)
32	0.426 (1.40	0.458 (1.50)	0.456 (1.50)	0.447 (1.47)	0.447 (1.47)	0.445 (1.46)
33	0.587 (1.93)	0.585 (1.92)	0.572 (1.88)	0.586 (1.92)	0.590 (1.94)	0.595 (1.95)
34	0.469 (1.54)	0.474 (1.56)	0.473 (1.55)	0.472 (1.55)	0.475 (1.56)	0.478 (1.57)
35	0.542 (1.78)	0.569 (1.87)	0.578 (1.90)	0.559 (1.83)	0.556 (1.82)	0.553 (1.81)
36	0.592 (1.94)	0.588 (1.93)	0.590 (1.94)	0.589 (1.93)	0.591 (1.94)	0.594 (1.95)
37	0.363 (1.19)	0.369 (1.21)	0.365 (1.20)	0.365 (1.20)	0.368 (1.21)	0.371 (1.22)
38	0.698 (2.29)	0.702 (2.30)	0.701 (2.30)	0.700 (2.30)	0.702 (2.30)	0.705 (2.31)
39	0.475 (1.56)	0.487 (1.60)	0.485 (1.59)	0.481 (1.58)	0.484 (1.59)	0.485 (1.59)
40	0.680 (2.23)	0.677 (2.22)	0.681 (2.23)	0.679 (2.23)	0.682 (2.24)	0.684 (2.24)
41	0.606 (1.99)	0.605 (1.98)	0.605 (1.98)	0.605 (1.98)	0.607 (1.99)	0.601 (2.00)
42	0.685 (2.25)	0.695 (2.28)	0.695 (2.28)	0.691 (2.27)	0.691 (2.27)	0.691 (2.27)
43	0.511 (1.68)	0.512 (1.68)	0.511 (1.68)	0.511 (1.68)	0.515 (1.69)	0.519 (1.70)

4 Wave Modeling

Life-cycle analysis of the disposal island designs for Poplar, James, and Barren Islands requires wave parameters around each island for a variety of storm conditions. Past studies have estimated waves for the study sites based on straight-line fetch wave generation with no near-island transformation. Straight-line fetch methods can underestimate wave heights and provide inaccurate wave direction in long, narrow water bodies, such as Chesapeake Bay (Smith 1991). Waves generated by a component of the wind down the axis of the bay may be larger than those in the direct wind direction. Neglecting wave transformation across shallow areas on boundaries of the bay, where the islands are located, can also lead to errors in local wave parameter distributions because of neglect of refraction, shoaling, and breaking processes.

The data processing and modeling steps required to convert wind and water level data into life-cycle analysis inputs appear in Figure 20. Winds were carefully validated and, in the case of the AES-40 hindcast winds, adjusted to compensate for reduced over-water drag. Three steps were required for wave modeling to produce the life-cycle inputs, including a restricted-fetch wave growth model (Smith 1991) in the Automated Coastal Engineering System (ACES), application of a parametric spectral shape in the Surface-water Modeling System (SMS), and the spectral transformation model STWAVE (Smith et al. 2001). This chapter presents a detailed description of this approach and is broken into five sections: winds for wave modeling, wave generation modeling, wave transformation modeling, model results, and summary.

Winds for Wave Modeling

Wind histories from the selected hurricanes and northeasters were used to generate open-bay wave estimates for transformation to shoreline locations at each study island. Two types of wind fields were used for the circulation modeling discussed in Chapter 3. For northeasters, wind fields were extracted from the AES-40 hindcast, and for hurricanes, wind fields were generated using the PBL model (details are provided in Chapter 2). For circulation modeling, the large-scale wind fields are generally of greatest importance, but for wave modeling in an enclosed area, the local winds are of greater significance.

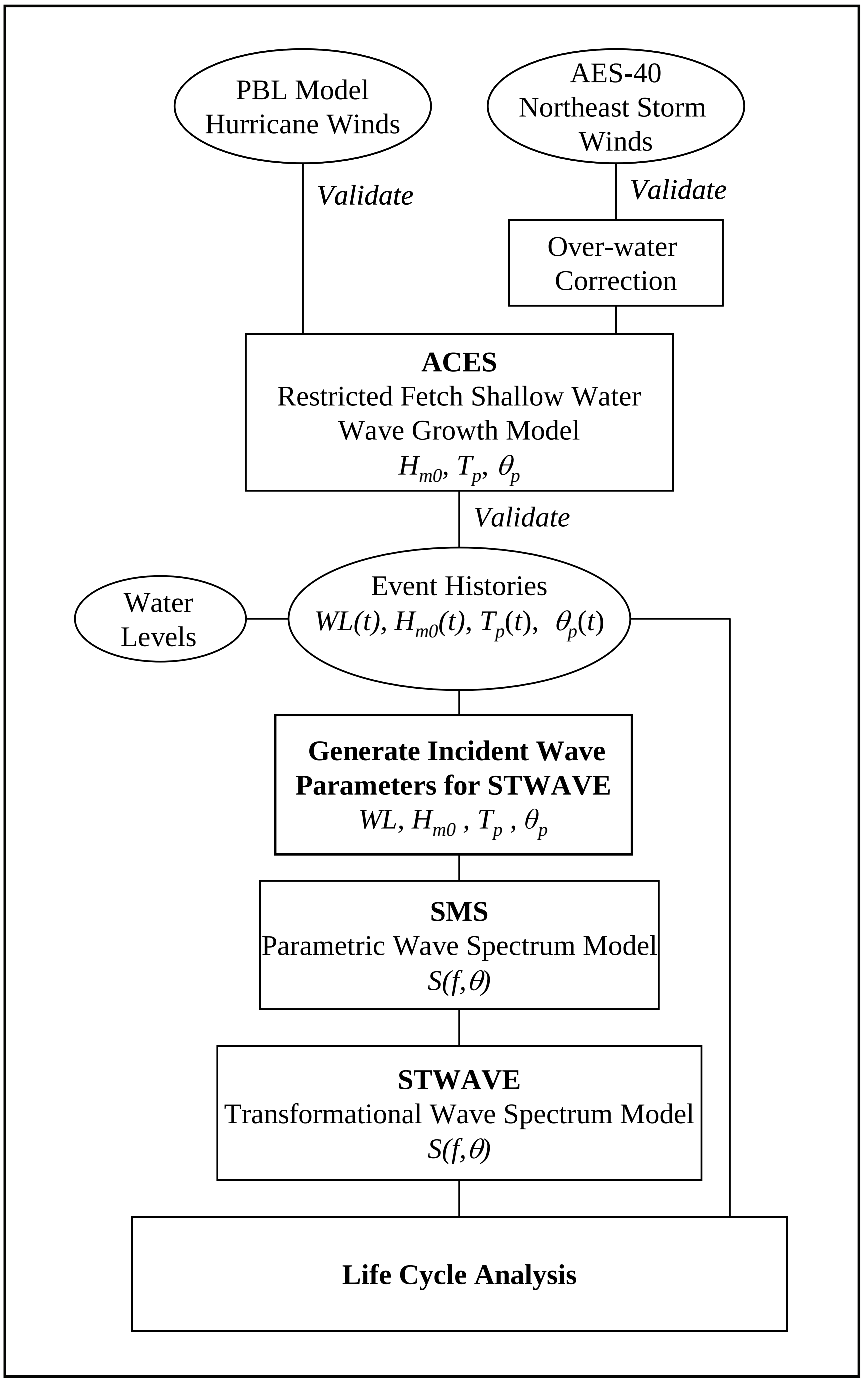


Figure 20. Wind and wave data processing flow chart

As with any modeling study, validation of input winds is essential to ensure optimal model performance. Both the PBL and AES-40 winds were compared with measurements from observation stations maintained by the National Climatic Data Center (NCDC), the National Data Buoy Center (NDBC), and the National Ocean Service (NOS). Locations of the most relevant stations and the three wind and wave prediction sites are provided in Figure 21. Also included is the location of an NOS Acoustic Doppler Current Profiler (ADCP) deployment that was used to validate the wave estimates (see the following section). Most of the meteorological stations in Figure 21 are in partially or fully sheltered environments and are not suitable for evaluating wind forcing conditions in the open bay. The NDBC station at Thomas Point (TPLM2), located in an open bay setting at lat. 38°53'54"N; long. 76°26'12"W, provides the best location to access wind fields for open-bay wave growth modeling. The longest meteorological time series in the region is from the NCDC Baltimore-Washington International (BWI) Station. However, this station is several miles inland and is located 47.6 m (156 ft) above sea level in an open field.

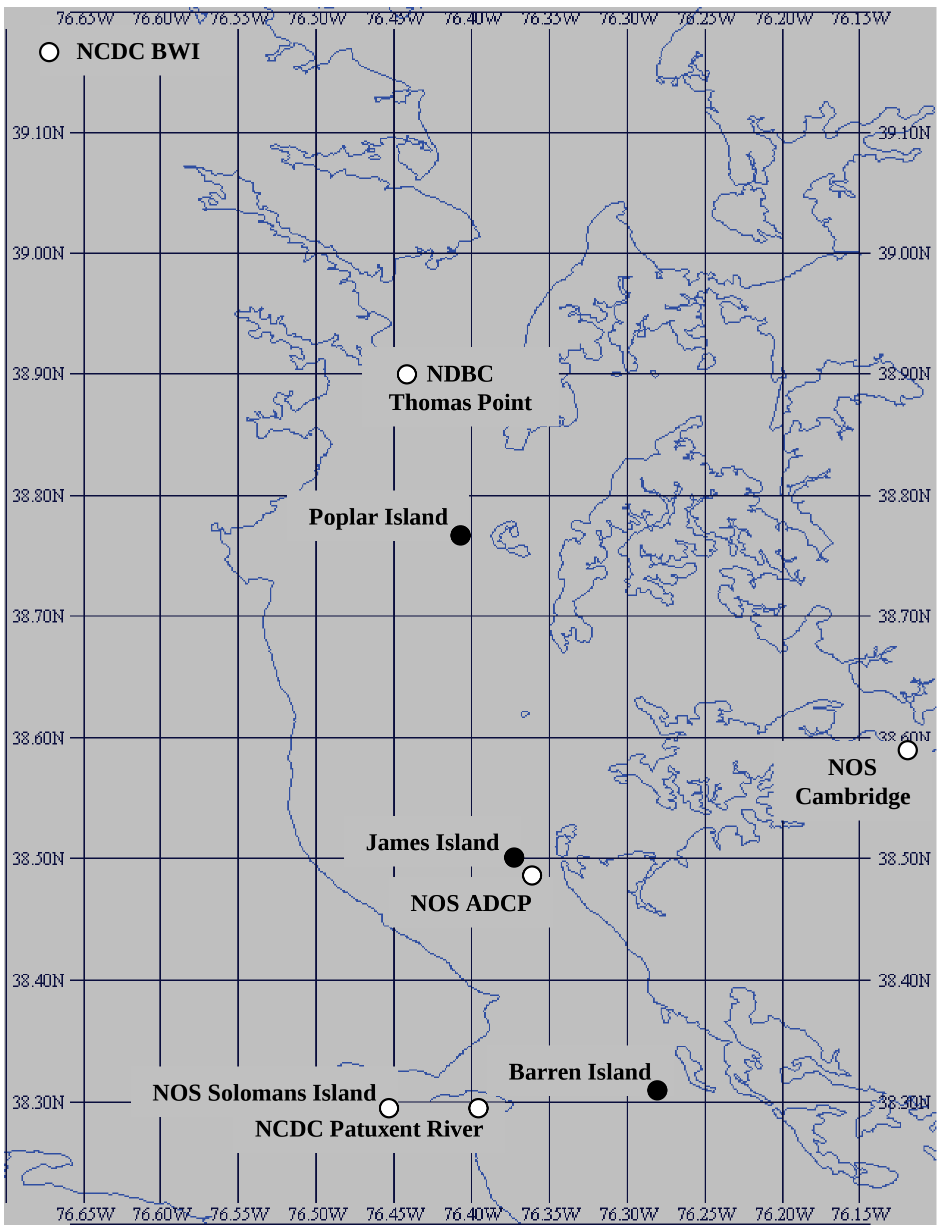
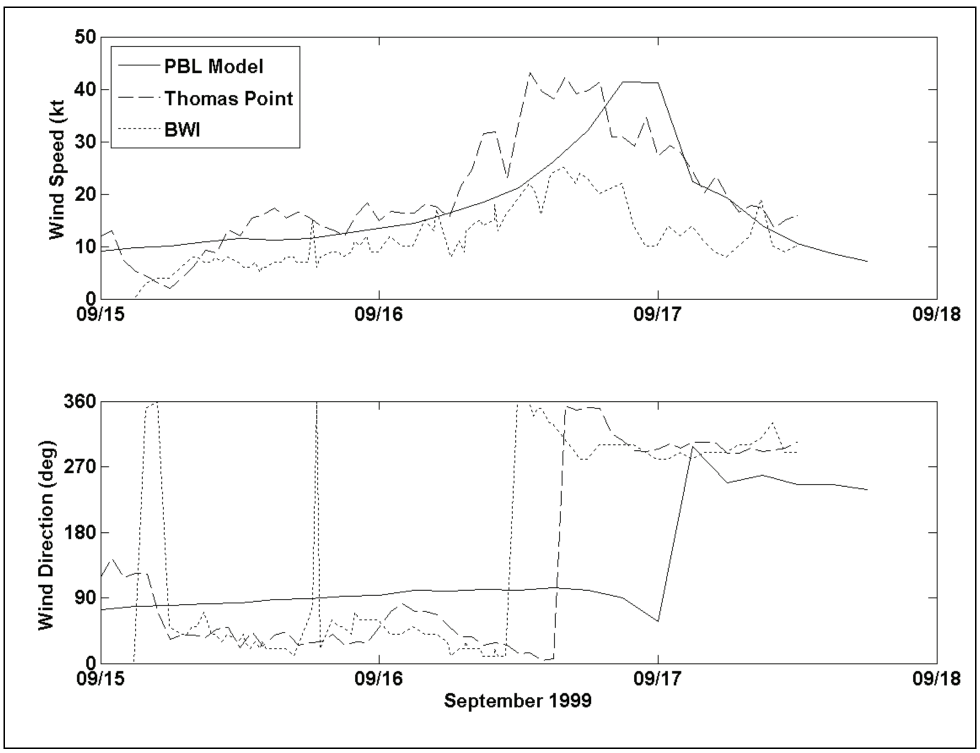


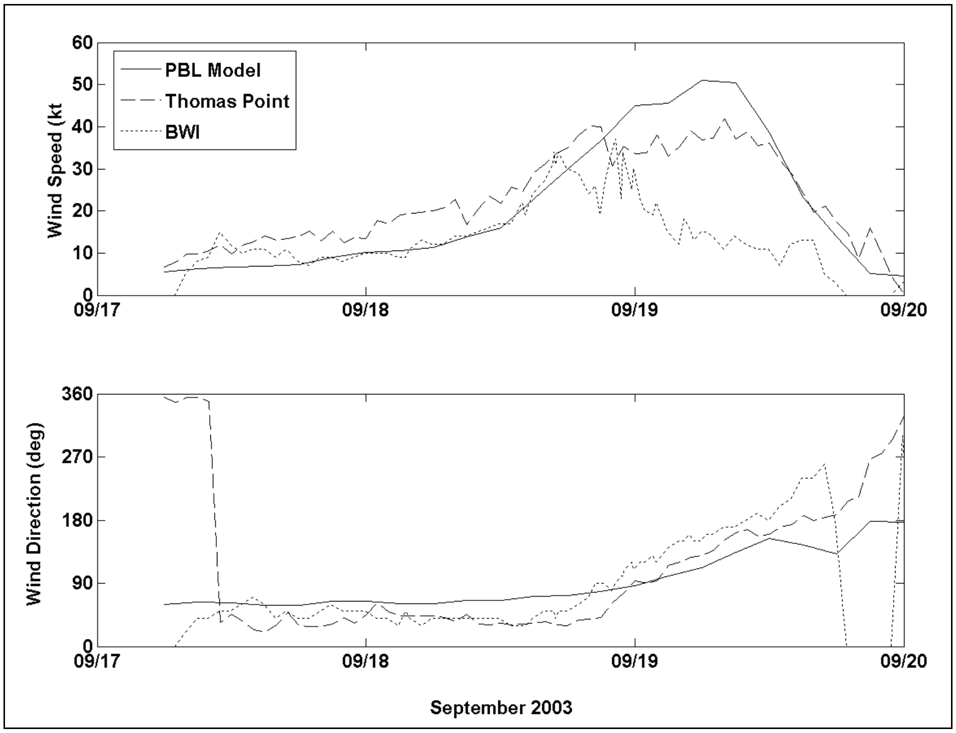
Figure 21. Locations of study islands and weather stations in Chesapeake Bay

The agreement between wind hindcasts and station measurements was highly variable from storm to storm. Comparisons of PBL model, Thomas Point station, and BWI station wind speed and direction histories during two major hurricanes appear in Figure 22. PBL estimates represent the time and duration of Hurricane Isabel quite well (Figure 22b), although maximum wind speeds are overestimated by approximately 5.1 m/sec (10 knots) and wind direction is off by approximately 45 deg. In contrast, the maximum wind speed and wind duration during Hurricane Floyd are matched very well by the PBL winds (Figure 22a), but with an 8-hr offset in the time of storm passage and approximately 60-deg offset in wind direction. Wind speeds at the overland BWI station are significantly reduced in strength compared to the open-bay observations. This wind-sheltering trend was also evident at the other stations depicted on Figure 21. Surprisingly, winds from BWI were used in earlier studies to generate wave climatologies at the mid-bay island sites (Kelley et al. 2002; Moffatt and Nichol 2002a, 2002b). Figure 22 suggests that the use of winds from a land-based or partially sheltered location for open-bay wave simulations will likely result in a significant underestimation of the actual wave climate.



a. Hurricane Floyd

Figure 22. Comparison of PBL hurricane winds with observations. (Wind speeds given in knots, to convert use 0.5144 m/sec/knot) (continued)



b. Hurricane Isabel

Figure 22. Concluded

The AES-40 hindcast winds required an over-water adjustment to match northeaster conditions in the open bay. Comparisons of AES-40 wind speed estimates to Thomas Point measurements during six major northeasters appear in Figure 23. Hindcast winds show reasonable agreement with observed winds below approximately 8.2 m/sec (16 knots). As wind speeds increase above 8.2 m/sec (16 knots), hindcast winds depict a linearly increasing negative bias that results in hindcast wind speeds 30 percent below the maximum observed winds. One possible explanation for this offset is that a coarse model resolution resulted in the application of a relatively high over-land drag coefficient to the bay region. This bias is removed from the AES-40 hindcast data with the following adjustment factor applied to winds ≥ 8.3 m/sec (16.5 knots):

$$U_{10}(\text{adjusted}) = 2.62 \text{ knots} * U_{10}(\text{original}) - 25.33 \text{ knots} \qquad (2)$$

The results of applying Equation 2 to the six northeasters discussed above appear in Figure 24. The hindcast results are significantly better. A linear least-squares regression through the data results in a slope that is within 10 percent of unity and a squared regression coefficient of $r^2 = 0.91$. A comparison of the original and adjusted AES-40 winds to Thomas Point observations during two typical northeasters appears in Figure 25. The wind directions and adjusted speeds match the mesoscale (day-to-day) variability quite well.

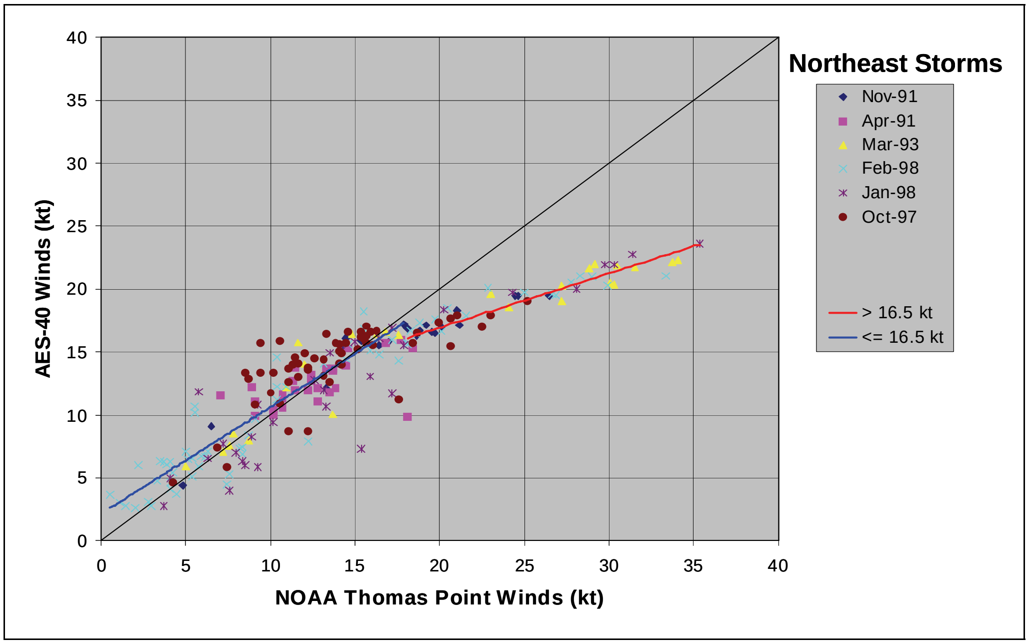


Figure 23. Scatter plot of AES-40 hindcast and Thomas Point station wind speeds during six major northeasters. A linearly increasing negative hindcast bias is evident at wind speeds above 8.2 m/sec (16 knots). (Wind speeds given in knots)

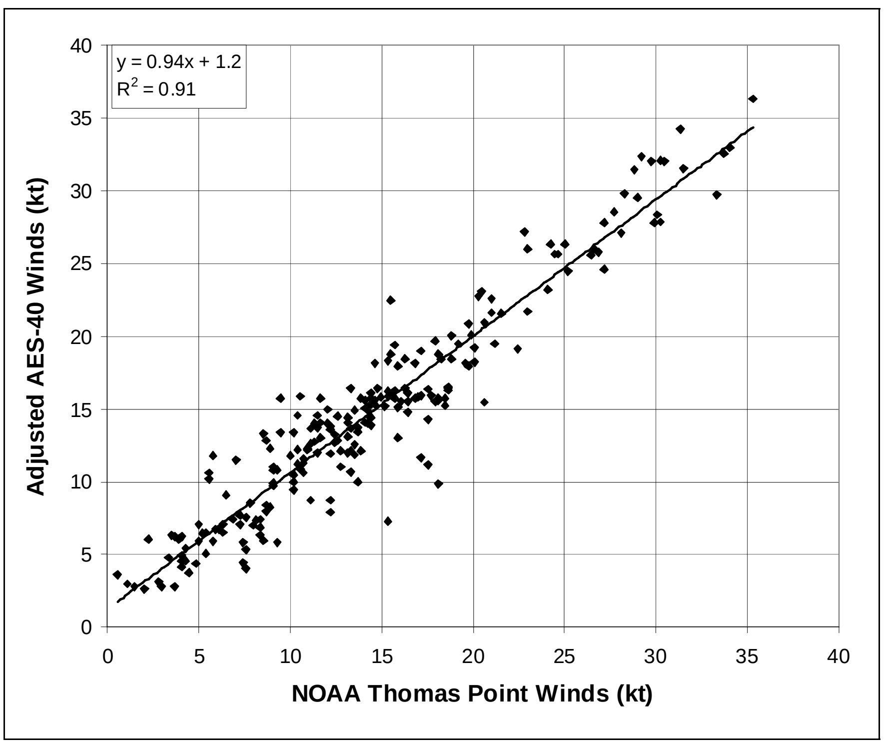


Figure 24. Scatter plot of adjusted AES-40 hindcast winds and Thomas Point measurements during six major northeasters. Linear adjustment of winds greater than 8.3 m/sec (16.5 knots) results in a reasonably good match between hindcast and observations. (Wind speeds given in knots)

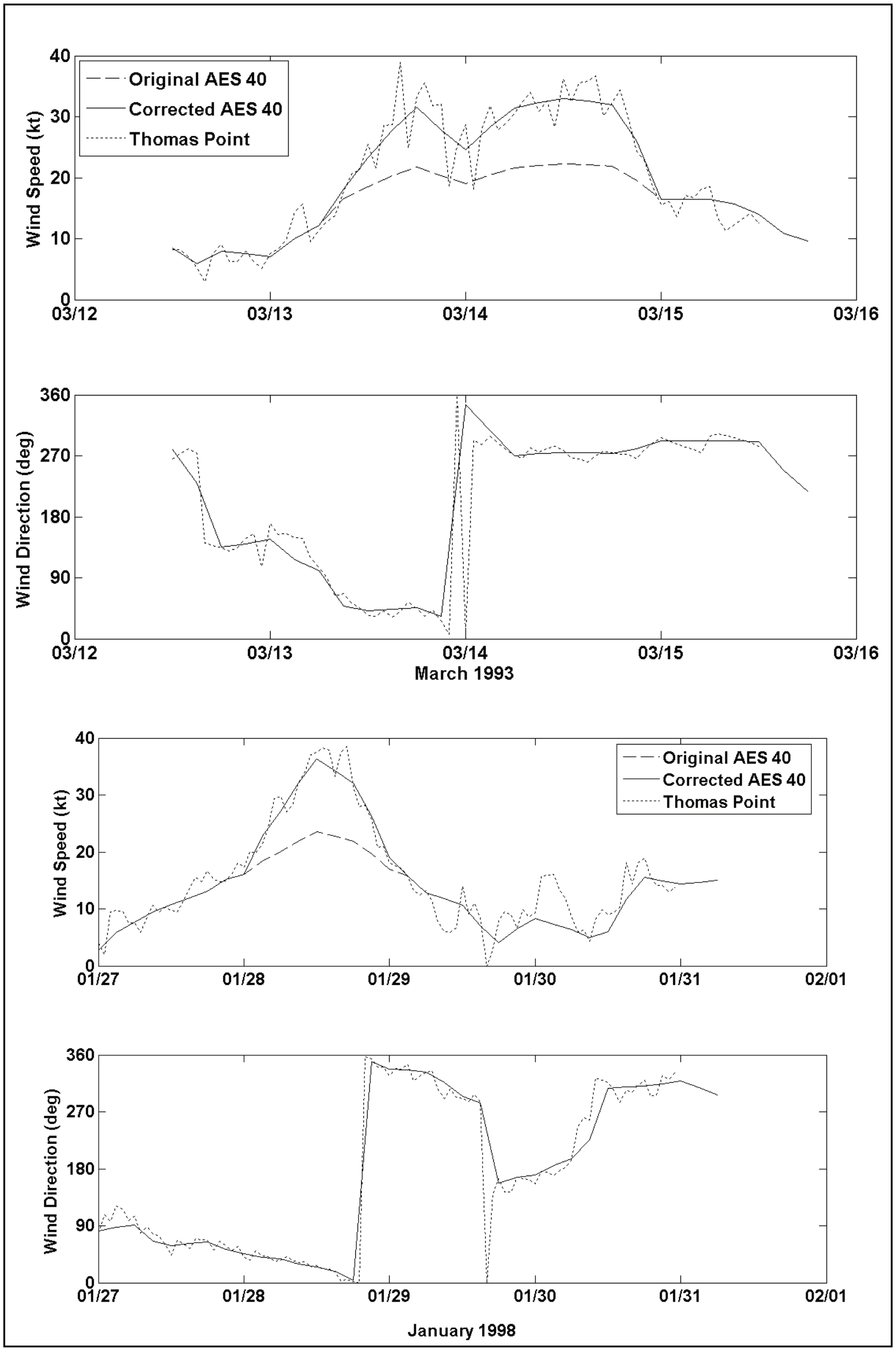


Figure 25. Evolution of original and adjusted AES-40 hindcast winds with observations during two northeasters. (Wind speeds given in knots)

Winds extracted from the PBL hurricane and adjusted AES-40 northeaster hindcasts provide reasonable inputs for estimating wave growth in the open bay. Wind forcing histories were generated offshore of each island location (Table 19). The following section addresses the application of these winds to estimating the evolution of surface wave conditions during each hurricane and northeaster.

Table 19
Offshore Wind and Wave Estimate Locations

Island	Latitude deg min sec	Longitude deg min sec	Approximate Water Depth m (ft)
Poplar	38 46 00 N	76 25 30 W	19.8 (65)
James	38 31 30 N	76 22 30 W	10.1 (33)
Barren	38 20 00 N	76 17 30 W	15.2 (50)

Wave Generation Modeling

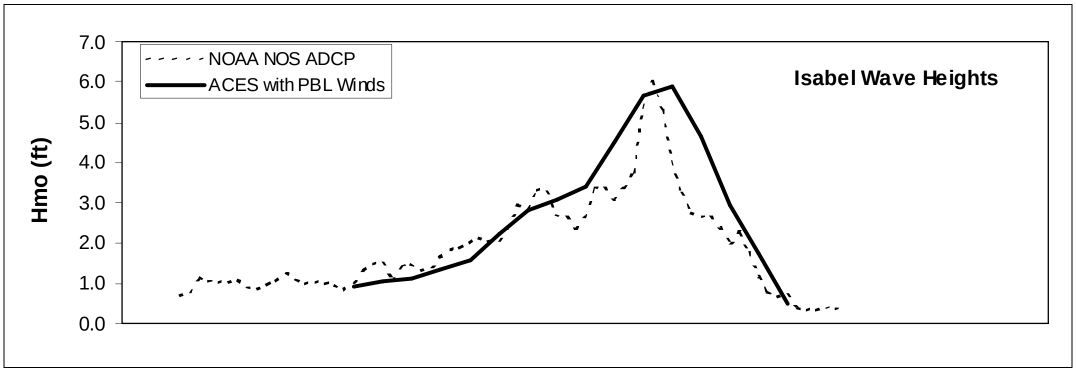
Hurricane and northeaster wave height (H_{mo}), peak period (T_p), and mean direction (θ_m) estimates were calculated at the "offshore" locations listed in Table 19 using the narrow-fetch methodology (Smith 1991) contained in ACES (Figure 20). At each site, fetch and average depth information at 10-deg directional increments were extracted from NOAA charts (U.S. Department of Commerce, Bathymetry of Chesapeake Bay, Plates 4 and 8). The resulting fetch and depth values for Poplar, James, and Barren Islands appear in Table 20. These fetches were input into ACES, along with the wind speed and direction time history for each of the storms. ACES outputs wave height, peak period, and mean direction.

Although the measurement of fetch is straightforward, estimating the average depth along a fetch is subjective, given the intense bathymetric variability within the bay. Hence, wave measurements collected during Hurricane Isabel by NOAA/NOS[1] were used to calibrate the selection of average depth by optimizing the agreement between ACES wave estimates and measurements at the NOS ADCP site (Figure 21). The results of these comparisons appear in Figure 26. The wave height estimates (Figure 26a) capture the overall duration of the storm and the wave height of 1.8 m (6 ft) at the storm peak. However the duration of high waves around the storm peak are overestimated by several hours. The wave period estimates (Figure 26b) do not match the high variability of the observations but do capture a general trend of increasing periods up to approximately 5.5 sec at the peak. Wave direction estimates (Figure 26c) show excellent agreement during the most intense part of the storm, with waves coming from the south.

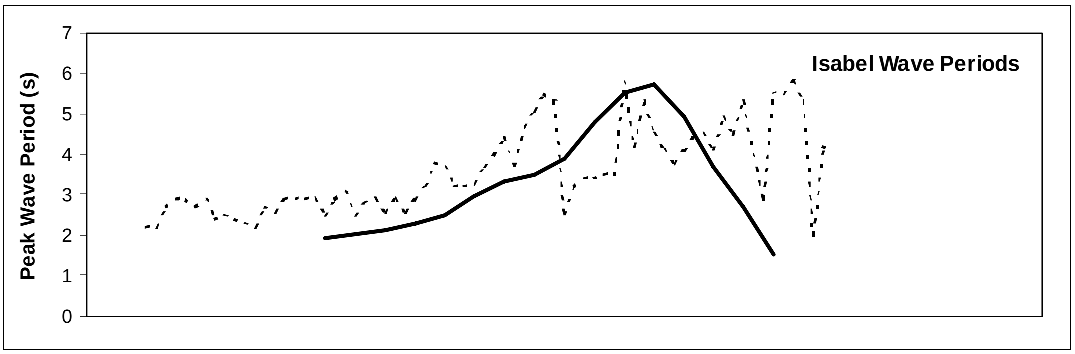
[1] ADCP data courtesy of H. H. Shih, Ph. D., P.E., NOAA/NOS, Silver Spring, MD.

Table 20
Fetch-Depth Table for ACES Wave Estimates

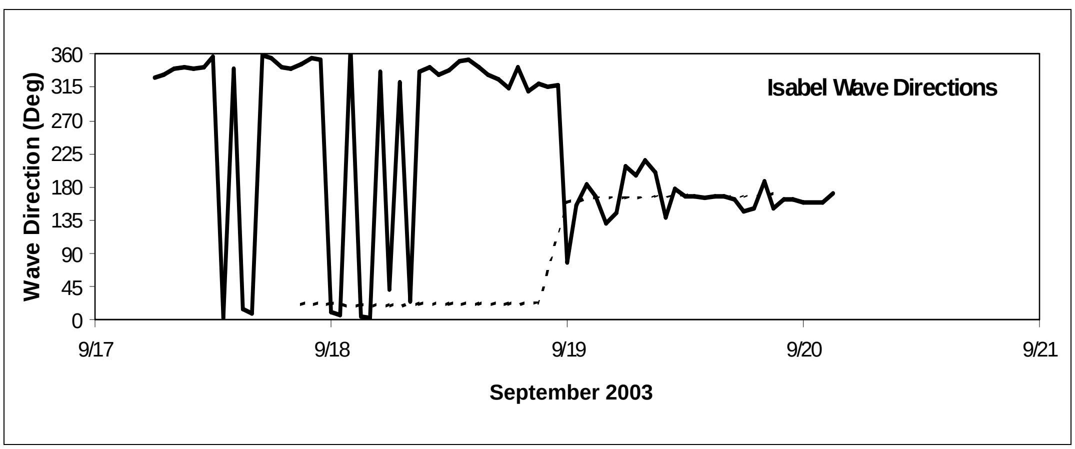
		Poplar Island		James Island		Barren Island	
Radial	Azimuth	Fetch, km (mi)	Depth, m (ft)	Fetch, km (mi)	Depth, m (ft)	Fetch, km (mi)	Depth, m (ft)
1	0	26 (16)	20 (66)	27 (17)	5 (16)	10 (6)	4 (13)
2	10	56 (35)	20 (66)	18 (11)	6 (20)	6 (4)	4 (13)
3	20	14 (9)	8 (26)	26 (16)	4 (13)	6 (3.5)	3 (10)
4	30	10 (6)	8 (26)	40 (25)	3 (10)	6 (3.5)	3 (10)
5	40	16 (10)	8 (26)	19 (12)	3 (10)	6 (3.5)	3 (10)
6	50	16 (10)	6 (20)	8 (5)	3 (10)	6 (3.5)	3 (10)
7	60	11 (7)	6 (20)	10 (6)	3 (10)	6 (3.5)	2 (7)
8	70	6 (4)	5 (16)	10 (6)	3 (10)	2 (1)	1 (3)
9	80	3 (2)	5 (16)	16 (10)	3 (10)	2 (1)	1 (3)
10	90	3 (2)	4 (13)	16 (10)	3 (10)	2 (1)	1 (3)
11	100	3 (2)	3 (10)	8 (5)	3 (10)	2 (1)	1 (3)
12	110	3 (2)	3 (10)	6 (4)	2 (7)	2 (1.5)	1 (3)
13	120	8 (5)	3 (10)	5 (3)	4 (13)	6 (4)	1 (3)
14	130	8 (5)	4 (13)	5 (3)	4 (13)	11 (7)	2 (7)
15	140	10 (6)	7 (23)	6 (4)	4 (13)	26 (16)	3 (10)
16	150	24 (15)	8 (26)	19 (12)	4 (13)	40 (25)	4 (13)
17	160	24 (15)	10 (33)	48 (30)	7 (23)	48 (30)	10 (33)
18	170	32 (20)	14 (46)	80 (50)	9 (30)	80 (50)	25 (82)
19	180	64 (40)	19 (62)	24 (15)	10 (33)	24 (15)	20 (66)
20	190	32 (20)	20 (66)	13 (8)	15 (49)	19 (12)	15 (49)
21	200	24 (15)	19 (62)	13 (8)	16 (52)	18 (11)	15 (49)
22	210	18 (11)	19 (62)	11 (7)	16 (52)	18 (11)	15 (49)
23	220	14 (9)	18 (59)	11 (7)	16 (52)	16 (10)	15 (49)
24	230	13 (8)	15 (49)	11 (7)	16 (52)	16 (10)	15 (49)
25	240	11 (7)	15 (49)	11 (7)	16 (52)	11 (7)	15 (49)
26	250	11 (7)	15 (49)	13 (8)	16 (52)	11 (7)	15 (49)
27	260	13 (8)	15 (49)	13 (8)	16 (52)	14 (9)	15 (49)
28	270	13 (8)	15 (49)	14 (9)	16 (52)	13 (8)	15 (49)
29	280	10 (6)	15 (49)	14 (9)	16 (52)	11 (7)	15 (49)
30	290	10 (6)	15 (49)	16 (10)	16 (52)	11 (7)	15 (49)
31	300	10 (6)	15 (49)	16 (10)	16 (52)	13 (8)	15 (49)
32	310	10 (6)	16 (52)	18 (11)	16 (52)	11 (7)	18 (59)
33	320	11 (7)	17 (56)	21 (13)	16 (52)	26 (16)	21 (69)
34	330	14 (9)	18 (59)	26 (16)	16 (52)	37 (23)	24 (79)
35	340	19 (12)	19 (62)	34 (21)	12 (39)	64 (40)	6 (20)
36	350	19 (12)	20 (66)	45 (28)	7 (23)	11 (7)	5 (16)



a. Isabel Wave Heights



b. Isabel Wave Periods



c. Isabel Wave Directions

Figure 26. Comparison of NOAA/NOS ADCP measurements with ACES wave estimates during Hurricane Isabel. PBL winds were used to drive the ACES estimates. (Wave height given in feet)

The hurricane and northeaster water levels described in Chapter 3 were synthesized with the ACES H_{mo}, T_p and θ_m estimates to generate 3-hourly wave and water level history files for each offshore location in Table 19. To reduce the computational demand for numerically simulating the transformation of these waves to various shoreline locations, a series of look-up tables was generated to cover the range of possible conditions at each site. The use of these look-up tables with STWAVE is described in the following section.

Wave Transformation Modeling

Numerical model simulations of wave transformation in Chesapeake Bay were required to provide the spatial and temporal variation of wave parameters around each of the three islands. This section describes the STWAVE model, model inputs, and sample model results. STWAVE was forced with directional wave spectra based on typical wave height, period, and direction combinations resulting from the wave generation modeling documented in the previous section. The simulations include representative tidal levels, which are required to simulate wave dissipation near the islands. The STWAVE simulations transformed waves resulting from northeasters and hurricanes in Chesapeake Bay.

STWAVE model description

The numerical model STWAVE (Smith et al. 2001) was used to transform waves to the project sites. STWAVE numerically solves the steady-state conservation of spectral action balance along backward-traced wave rays:

$$\begin{aligned}&(C_{ga})_x \frac{\partial}{\partial x}\frac{C_a C_{ga}\cos(\mu-\alpha)E(f,\alpha)}{\omega_r} + \\ &(C_{ga})_y \frac{\partial}{\partial y}\frac{C_a C_{ga}\cos(\mu-\alpha)E(f,\alpha)}{\omega_r} = \sum\frac{S}{\omega_r}\end{aligned} \qquad (3)$$

where

C_{ga} = absolute wave group celerity

x,y = spatial coordinates, subscripts indicate x and y components

C_a = absolute wave celerity

μ = current direction

α = propagation direction of spectral component

E = spectral energy density

f = frequency of spectral component

ω_r = relative angular frequency (frequency relative to the current)

S = energy source/sink terms

The source terms include wind input, nonlinear wave-wave interactions, dissipation within the wave field, and surf-zone breaking. The terms on the left-hand side of Equation 3 represent wave propagation (refraction and shoaling), and the source terms on the right-hand side represent energy growth or decay in the spectrum.

The assumptions made in STWAVE are as follows:

a. Mild bottom slope and negligible wave reflection.

b. Spatially homogeneous offshore wave conditions.

c. Steady waves, currents, and winds.

d. Linear refraction and shoaling.

e. Depth-uniform current.

f. Negligible bottom friction.

STWAVE is a half-plane model, meaning that only waves propagating toward the coast are represented. Waves reflected from the coast or waves generated by winds blowing offshore are neglected. Wave breaking in the surf zone limits the maximum wave height based on the local water depth and wave steepness:

$$H_{mo_{\max}} = 0.1L \tanh kd \tag{4}$$

where

H_{mo} = zero-moment wave height

L = wavelength

k = wave number

d = water depth

STWAVE is a finite-difference model and calculates wave spectra on a rectangular grid with square grid cells. The model outputs zero-moment wave height, peak wave period (T_p), and mean wave direction (α_m) at all grid points and two-dimensional spectra at selected grid points.

Wave model inputs

The inputs required to execute STWAVE are as follows:

a. Bathymetry grid (including shoreline position and grid size and resolution).

b. Incident frequency-direction wave spectrum on the offshore grid boundary.

c. Current field (optional).

d. Tide elevation, wind speed, and wind direction (optional).

Bathymetry grids. For each island, several bathymetry grids were required to model the wave transformation. The same underlying bathymetry was used for each grid, but the grid orientation was changed so that the input wave direction was less than 60 deg relative to the x-axis of the grid. The grid specifications for each island are given in Table 21. The grid origin is given in

Maryland State Plane coordinates. The grid orientation is the orientation of the grid x-axis measured counter-clockwise from east (SMS default). The grid naming convention indicates the island and the approximate incident wave direction. The bathymetry for each grid is a compilation of the GEODAS data and survey data provided by the Baltimore District. Depths are relative to mllw. Figures 27 to 29 show examples of the grids for each island.

Table 21
Bathymetry Grid Specifications

Grid	X Origin, m (ft)	Y Origin, m (ft)	Δx, m (ft)	Orientation, deg	X Cells	Y Cells
Poplar NW	446,608 (1,465,250)	118,192 (387,770)	46 (150)	330	166	280
Poplar NE	455,518 (1,494,480)	131,405 (431,120)	46 (150)	220	210	260
Poplar N	445,307 (1,460,980)	128,449 (421,420)	46 (150)	290	292	202
Poplar S	457,307 (1,500,350)	117,318 (384,900)	46 (150)	90	170	220
James W	452,847 (1,485,720)	88,200 (289,370)	46 (150)	0	150	307
James S	457,965 (1,502,510)	85,778 (281,425)	46 (150)	75	270	193
James NW	447,222 (1,467,265)	101,781 (333,926)	46 (150)	287	280	230
James NE	454,301 (1,490,490)	103,882 (340,820)	46 (150)	240	216	230
Barren NW	455,502 (1,494,430)	73,836 (242,245)	46 (150)	337	245	182
Barren SE	466,435 (1,530,300)	60,408 (198,190)	46 (150)	80	325	232
Barren W	463,366 (1,520,230)	66,962 (219,690)	46 (150)	20	101	225

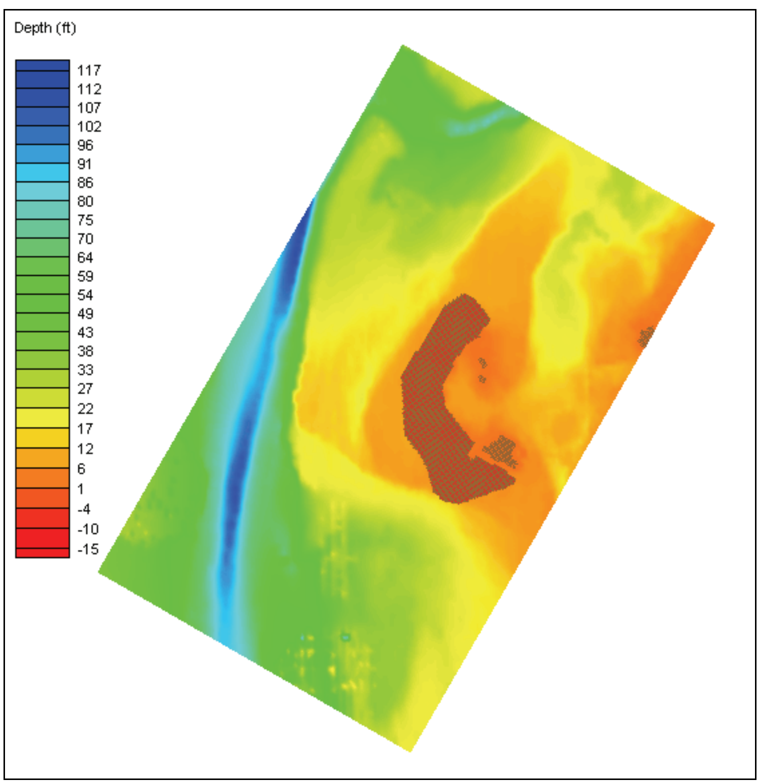


Figure 27. Poplar northwest bathymetry grid (depths in feet mllw). Land is shown in brown. (To convert from feet to meters, use 0.3048 m/ft)

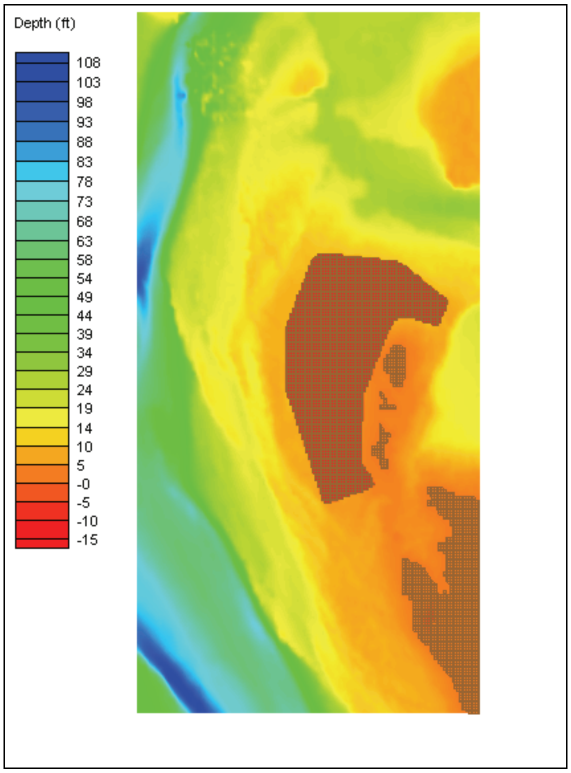


Figure 28. James west bathymetry grid (depths in feet mllw). Land is shown in brown

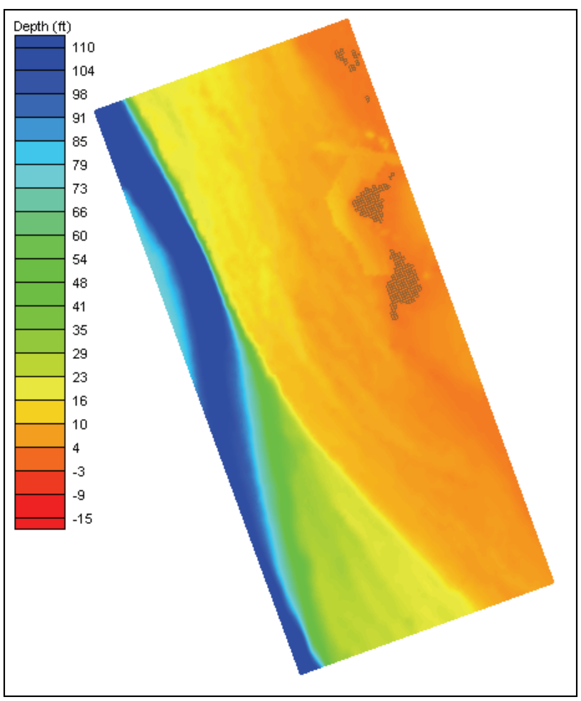


Figure 29. Barren west bathymetry grid (depths in feet mllw). Land is shown in brown

Input wave spectra. Input wave spectra are required to drive STWAVE on the "offshore" grid boundary. The definition of "offshore" changes for each grid, and it is the boundary across which the waves are propagating. The wave generation model provides only wave height, peak wave period, and mean wave direction, so parametric spectral shapes are used to generate the input spectra. The wave energy is distributed in frequency using the TMA spectral shape with a spectral peakedness parameter of 3.3 (Bouws et al. 1985) and in direction using a $\cos^4(\alpha-\alpha_m)$ distribution, where α_m is the mean wave direction. The input spectra have 30 frequencies, starting with 0.04 Hz and incrementing by 0.01 Hz. The directional resolution is 5 deg. The wave parameters run for each grid are summarized in Table 22.

Water level. The range of water levels (combination of tide and storm surge) was determined by the circulation model simulations (Chapter 3). Water levels that occurred in the target storms in combination with the incident waves were modeled. The water levels run for each grid and associated wave conditions are given in Table 22. Water level is applied in STWAVE as constant water depth change over the grid.

Winds and currents. Wind and current effects were not included within the STWAVE domains.

Table 22
Waves and Water Levels Simulated in STWAVE

Grid	Depth Grid Boundary, m (ft)	Grid Shore Normal, deg	Wave Angle, deg	Water Levels, m (ft) mllw	Wave Height, m (ft)	Wave Period, sec
Poplar NW	20 (66)	300	265, 335	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4)	3
Poplar NE	10 (33)	50	50	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5, 7
Poplar N	40 (131)	340	10	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5, 7
Poplar S	20 (66)	180	150, 180	0 (0), 0.3 (1), 0.6 (2), 0.9 (3), 1.2 (4), 1.5 (5), 1.8 (6), 2.1 (7)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5, 7, 9
James W	30 (98)	270	270	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5
James S	30 (98)	195	165	0 (0), 0.3 (1), 0.6 (2), 0.9 (3), 1.2 (4), 1.5 (5)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5, 7, 9
James NW	20 (66)	343	343	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5
James NE	10 (33)	30	30	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5
Barren NW	38 (125)	293	338	0 (0), 0.3 (1), 0.6 (2), 0.9 (3), 1.2 (4), 1.5 (5)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5, 7
Barren SE	35 (115)	190	145, 167	0 (0), 0.3 (1), 0.6 (2), 0.9 (3), 1.2 (4), 1.5 (5)	0.6 (2), 1.2 (4), 1.8 (6), 2.4 (8)	3, 5, 7
Barren W	33 (108)	250	227, 262	0 (0), 0.3 (1), 0.6 (2), 0.9 (3)	0.6 (2), 1.2 (4)	3

Model Results

Some sample STWAVE results are shown is this section to illustrate the range of conditions simulated. The wave periods within Chesapeake Bay are relatively short, but periods as long as 9 sec were hindcast for extreme cases. Figure 30 shows the wave height and direction for two simulations on the Poplar South grid. The top panel is a period of 3 sec and the bottom panel is 9 sec, both for incident H = 1.8 m (6 ft), dir = 180 deg, and water level = 0 m (0 ft). The longer-period waves interact more strongly with the bottom, resulting in greater refraction (turning of the wave directions) and shoaling (increases in wave height in shallow depths). To the west of the proposed island, wave heights are approximately 0.3 m (1 ft) higher for the longer-period wave in the shallow areas. The wave vectors also show more turning of the wave direction toward the island for the longer-period wave.

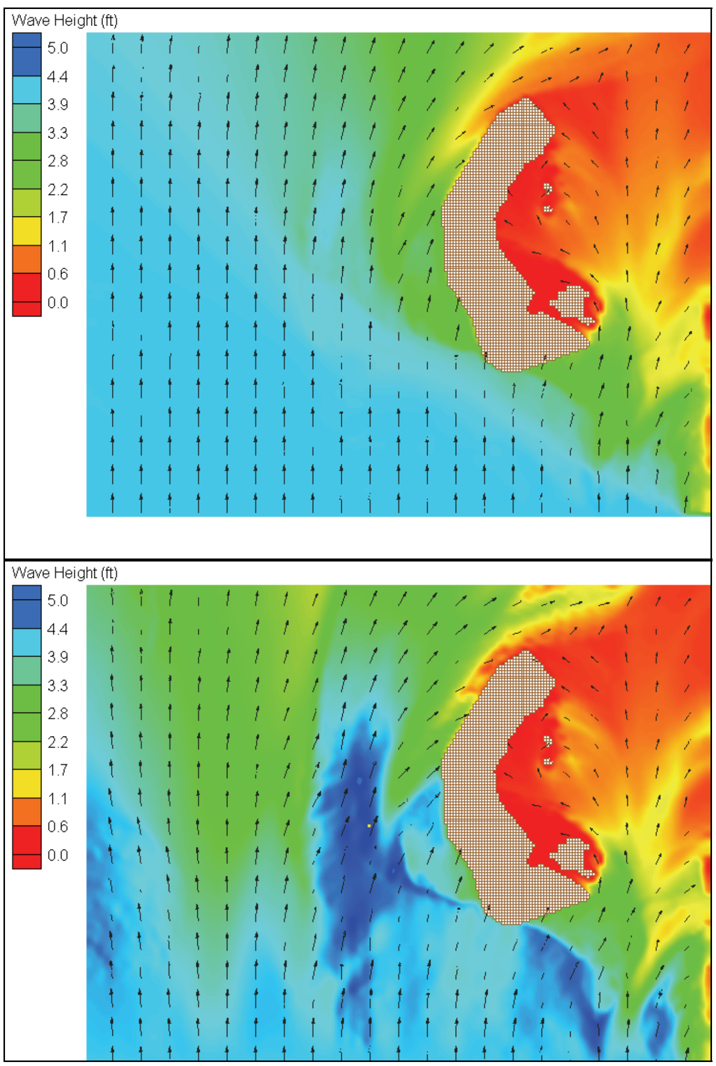


Figure 30. Poplar south grid wave height contours for incident waves H = 1.8 m (6 ft), T = 3 sec (top) and 9 sec (bottom), dir = 180 deg, and water level = 0 m mllw

Wave height is a key parameter for designing the island revetment because it is raised to a power greater than 1. The incident wave height from the generation modeling may be altered significantly through transformation before reaching the islands and may vary along an island. Figure 31 shows a simulation for James Island with waves from the west and incident wave heights of 0.6 m (2 ft) and 2.4 m (8 ft) and a period of 5 sec. Near the island, the 0.6-m (2-ft) wave shoals to a height of approximately 0.67 m (2.2 ft) and the 2.4-m (8-ft) wave breaks and is dissipated to approximately 1.1 m (3.5 ft). If waves are depth-limited, then a larger offshore wave height may not translate to a larger nearshore wave height. Similarly, when the water depth around the island is increased because of storm surge and tide, larger waves may attack the island. Figure 32 shows results from Barren Island with waves from the west and a modest increase in water level from 0 to 1 m (0 to 3 ft) mllw (H = 1.2 m (4 ft), T = 3 sec). Wave heights near Barren increase from approximately 0.76 m (2.5 ft) (depth limited) to 1.2 m (4 ft) in some areas with the increase in water level.

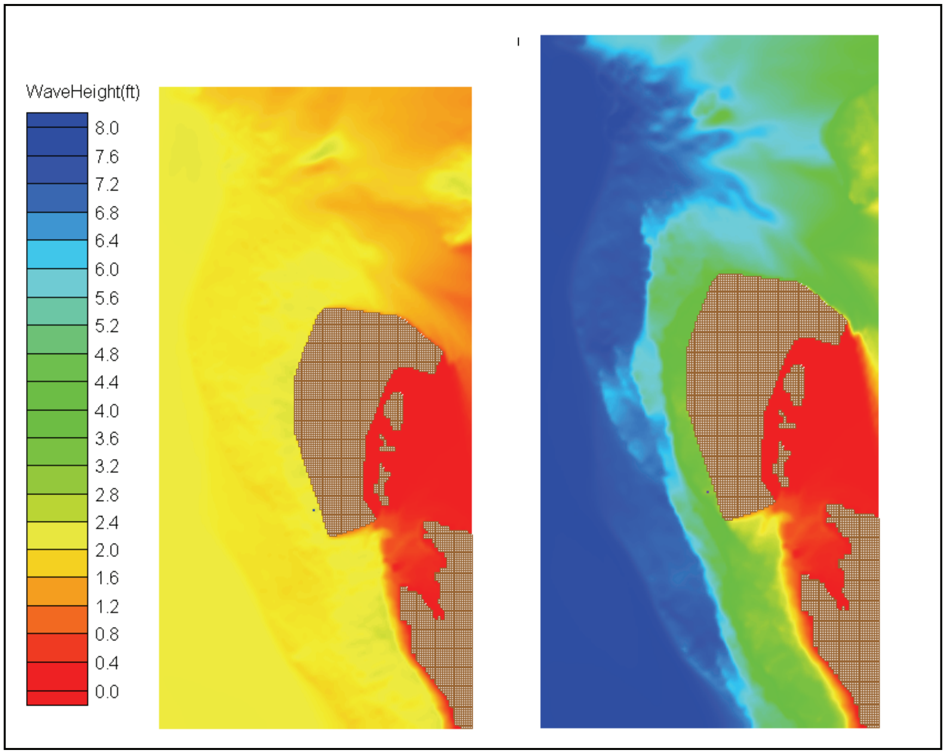


Figure 31. James west grid wave height contours for incident waves H = 0.61 m (2 ft) (left) and 2.4 m (8 ft) (right), T = 5 sec, dir = 270 deg, and water level = 0 m mllw

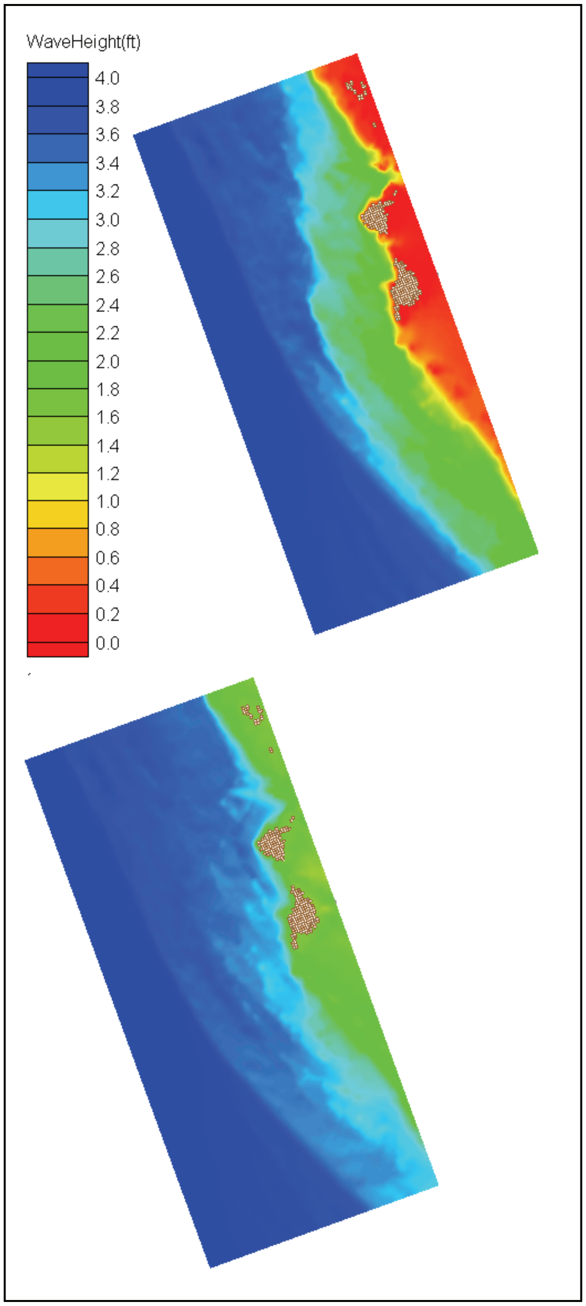


Figure 32. Barren west grid wave height contours for incident waves H = 1.2 m (4 ft), T = 3 sec, dir = 262 deg, and water level = 0 ft mllw (left) and +1 m (3 ft) mllw (right)

Summary of Wave Results

Modeling of waves at Poplar, James, and Barren Islands required a three-step process: generation of wave parameters (height, period, and direction) using the narrow-fetch methodology with wind speed and direction, application of the TMA parametric spectral shape to estimate wave spectra from the wave parameters, and application of the STWAVE wave model to calculate the transformation of the waves over the complex nearshore bathymetry at each site. Accurate wind input is critical to wave modeling. Ideally, wind measurements would be used to drive the wave model, but open-water measurements were not available over sufficient years. Thus, hurricane winds were hindcast using the PBL model and northeaster winds were extracted from the AES-40 hindcast (Chapter 2). Winds for wave modeling were validated with open-water measurements at the NDBC Thomas Point station. The PBL winds showed some lags in time, but generally gave good agreement in the peak parameters. The AES-40 winds underestimated high wind speeds (>8.5 m/sec or >16.5 knot), and a correction factor was developed and applied.

The narrow-fetch technology was validated for Chesapeake Bay using wave measurements from Hurricane Isabel. These measurements were used to calibrate the water depth input for the generation modeling. Wave generation was modeled with the narrow-fetch methodology in ACES. The model input includes radial fetch lengths (10-deg increments) and wind speed and direction time history. The output is a time history of wave heights, periods, and directions. The range of these storm wave parameters was wave heights of 0.6 m to 2.4 m (2 to 8 ft) and wave periods of 3-9 sec. The maximum values varied based on wave direction. Preferential wave directions for each island are noted in Table 22. These directions coincide with the longer fetches in the bay. Waves are generated along these fetches by the component of the wind in the fetch direction.

For each island, the representative wave parameters and range of water levels (Chapter 3) were used to drive STWAVE. A parametric spectral wave shape was applied to estimate wave spectra from the wave parameters. These spectra were input to STWAVE with the water levels. STWAVE calculates the wave shoaling, refraction, sheltering, and breaking to give the spatial distribution of wave height, period, and direction around each island. Because of the complex bathymetry and multiple wave angles, several model grids were required for each island (Table 21). Water level is a critical parameter in the transformation because of the shallow depths around the islands. For depth-limited conditions, the wave height varies linearly with water depth. Results from the STWAVE simulations were stored in look-up tables. These tables are a matrix of local wave parameters around each island that coincide to an input wave height, wave period, wave direction, and water level combination. With these tables, time histories of the "offshore" storm waves can be converted to transformed nearshore wave parameters for application of the life-cycle analysis for design of island revetment.

5 Life-Cycle Simulation Methodology – Waves and Water Levels

This chapter describes the procedures used for life-cycle simulation of waves and water levels. Methods used to develop a 148-year time history of historical storm events and return period wave and water levels at nearshore stations around Poplar Island are described in the first two sections. The same methods used to determine waves and water levels around James and Barren Islands are described in the following two sections. Procedures for using the ELS to create future wave and water level life-cycle scenarios are described in the final section. These procedures were applied to create a large number of possible future 50-year life cycles that are statistically consistent with historical information. The methods used to optimize the design of Poplar, James, and Barren Islands protective structures are presented in Chapters 7, 8, and 9, respectively.

Sequence of Historical Storms

The life-cycle simulation approach begins with a known wave and water level time history over a multi-year time period. Initially, the time history is based on historical data or hindcasts. The time period covered by tropical storms is 148 years (1856-2003), while the time period covered by northeasters is only 50 years (1954-2003). Northeasters are more common than tropical storms and are less likely than tropical storms to be atypically severe. The 50-year period of northeasters available in the hindcasts is expected to give a good representation of the range of northeasters affecting the project areas. To populate the early tropical storm years with northeasters, the northeasters were folded back as shown in Table 23. Care was taken to fold leap years back into leap years and similarly with non-leap years. Thus, a 148-year offshore time history of historical storm waves and water levels was created. The final time history contains 179 storms.

The time history of storms was padded with quiescent waves. The quiescent waves were measured in the mid-bay area continuously during 2003. In addition, water levels measured in the bay near each island were incorporated into the time history file. These measured waves and water levels were folded back into all years in between storm events in order to create a continuous and realistic historical time history of waves and water levels covering 148 years at 3-hr intervals. Cumulative Julian day was added, starting with 0.000 at 0000 hrs, 1 January 1856, and accumulating through the 148 years.

Table 23
Northeaster Years Matched to Early Tropical Storm Years

Tropical Storm Years	Matched Northeaster Years
1856 (leap year) - 1857	2000 (leap year) - 2001
1858 – 1905	1954 – 2000 (leap year) & 2002 (nonleap year)
1906 – 1953	1954 – 2000 (leap year) & 2003 (nonleap year)

Design Waves and Water Levels for Poplar Island

Time history of storm waves and water levels

The next step is to transform the 148-year offshore wave time history to selected points along the study area coast. For Poplar Island, sixteen points (sta 1-16) were selected adjacent to shore for design analysis (Figure 33). Another 16 points (sta 17-32) were selected farther offshore from each of the points shown, approximately 300 m from shore, but these are not included in the figures. Finally, seven points (sta 33-39) were selected adjacent to shore for the planned expansion of the north end of Poplar Island (Figure 34). Reference water depths (mllw) at the stations are given in Table 24. These depths are based on the Baltimore District's most recent survey data, which is more recent than the bathymetry used for water level and wave numerical model grids. The computer program that transforms the 148-year offshore time history to nearshore design analysis stations is Poplar_timehist.f.

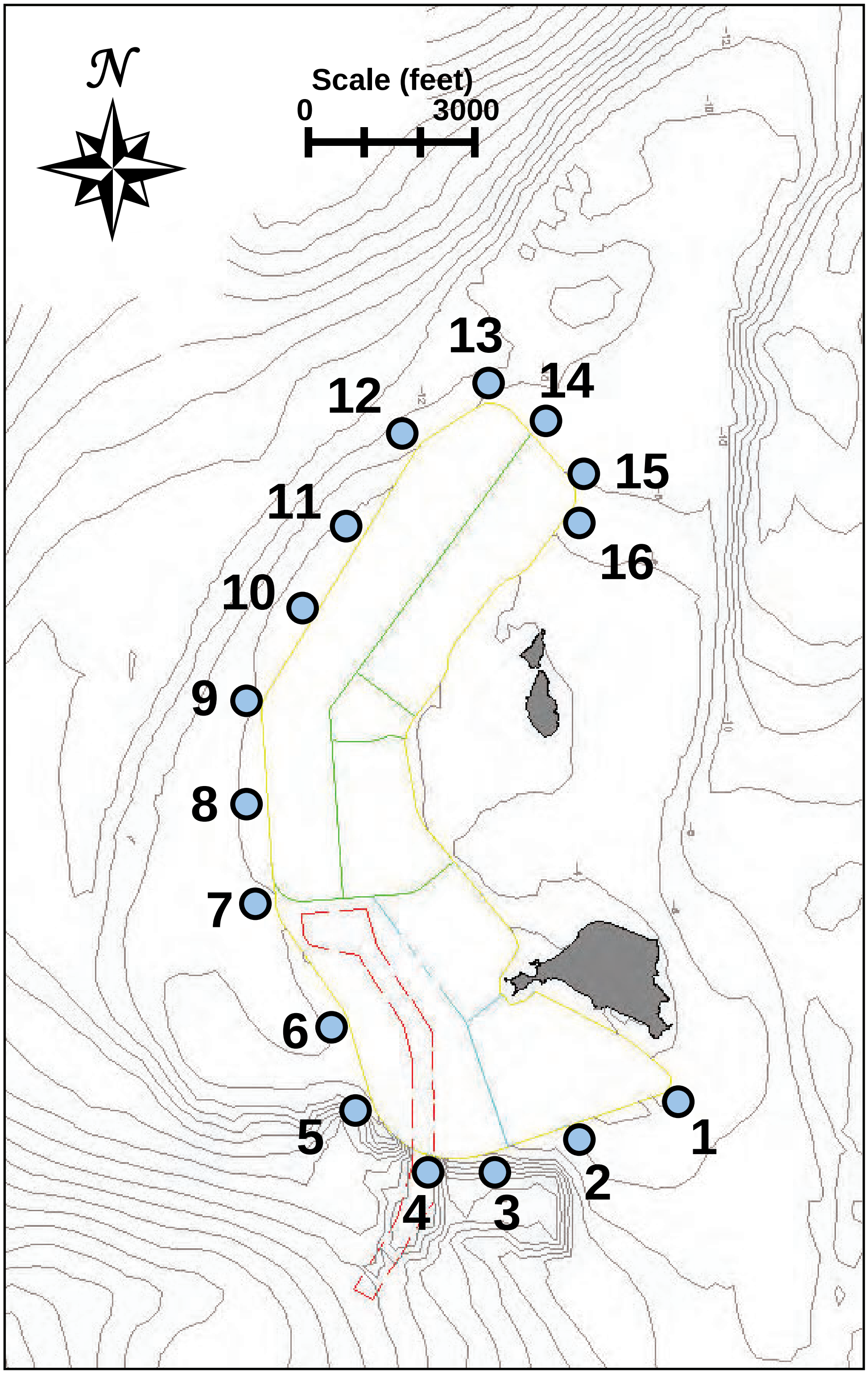


Figure 33. Design analysis stations, Poplar Island, as it existed in 2005 (contours show bathymetry at 2-ft intervals)

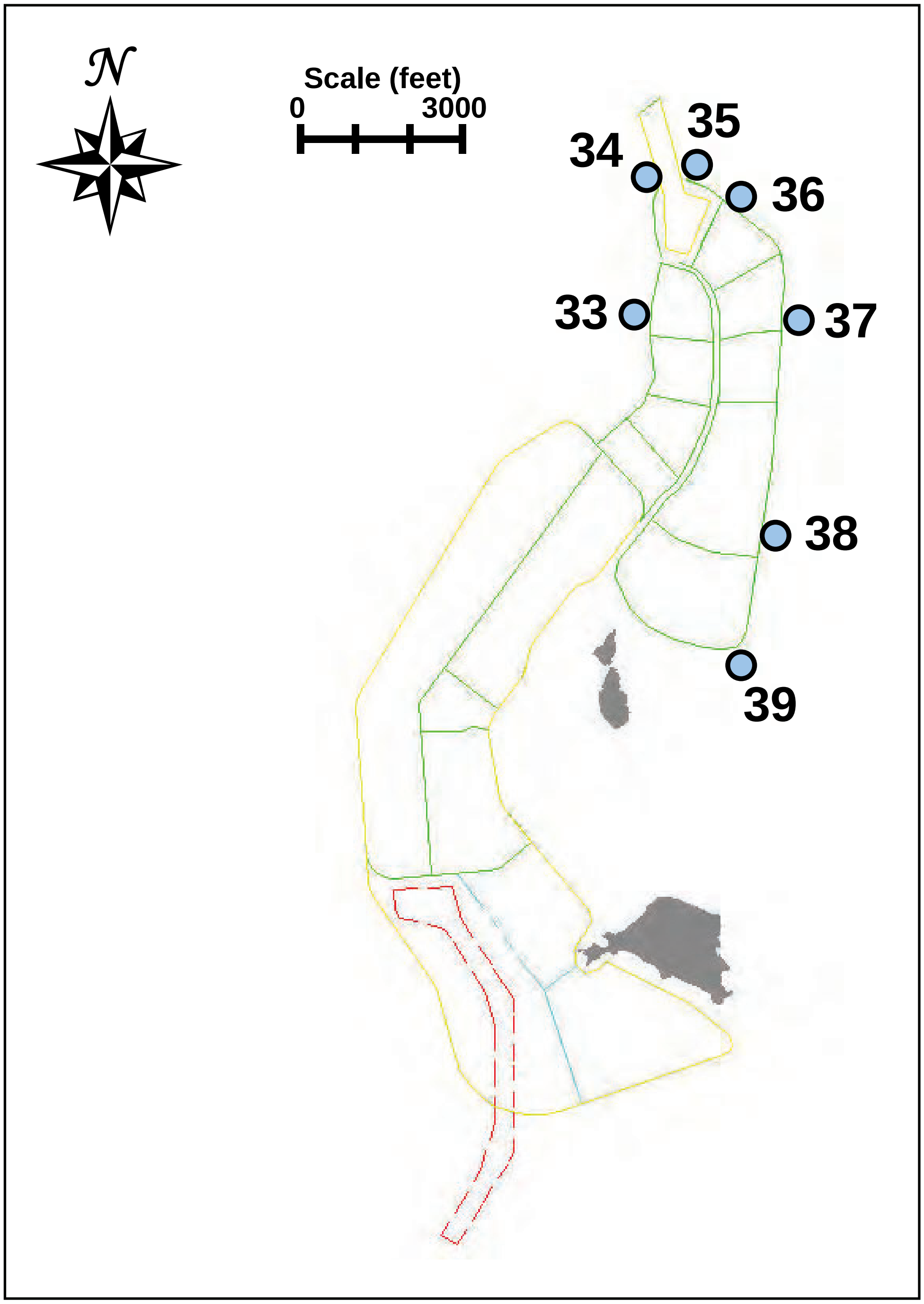


Figure 34. Design analysis stations, Poplar Island, planned expansion

Table 24
Water Depths at Poplar Island Design Analysis Stations

Station No.	Depth, m (ft) mllw	Station No.	Depth, m (ft) mllw
1	1.40 (4.59)	21	5.49 (18.01)
2	1.83 (6.00)	22	2.44 (8.01)
3	3.05 (10.01)	23	2.44 (8.01)
4	4.45 (14.60)	24	2.65 (8.69)
5	2.74 (8.99)	25	3.05 (10.01)
6	2.29 (7.51)	26	2.90 (9.51)
7	2.59 (8.50)	27	3.05 (10.01)
8	2.29 (7.51)	28	3.66 (12.01)
9	2.44 (8.01)	29	3.81 (12.50)
10	2.13 (6.99)	30	2.90 (9.51)
11	2.29 (7.51)	31	2.59 (8.50)
12	3.05 (10.01)	32	1.98 (6.50)
13	3.05 (10.01)	33	3.35 (10.99)
14	2.74 (8.99)	34	3.50 (11.48)
15	2.44 (8.01)	35	3.35 (10.99)
16	2.13 (6.99)	36	3.05 (10.01)
17	1.52 (4.99)	37	3.05 (10.01)
18	1.83 (6.00)	38	2.44 (8.01)
19	5.00 (16.40)	39	1.68 (5.51)
20	5.28 (17.32)		

The offshore wave and water level time history was transformed to the selected nearshore stations using look-up table information from the STWAVE runs discussed in Chapter 4. The look-up table provided a transformation factor to relate offshore and nearshore significant wave heights and similar factors for wave period and direction. Factors from the STWAVE case that best matched the offshore wave case were used for period and direction. For significant wave height, the factor was interpolated from STWAVE cases that best matched the offshore significant height, direction, and water level and bracketed the offshore wave period. Water level in the nearshore station time history was taken from the closest ADCIRC nearshore save station (Chapter 3) and converted from msl to mllw datum.

Since STWAVE was run only with the existing Poplar Island, waves extracted for stations around the proposed north expansion needed to be screened to account for sheltering effects of the expansion land mass. The screening was applied based on wave direction from STWAVE at each station. Directions accepted at each station are given in Table 25. For cases with directions outside the accepted range, significant height was reduced to 0.01 m (0.03 ft).

Table 25
Nearshore Wave Directions Accepted at Stations Around Planned Poplar Island Expansion

Station No.	Direction Range (deg azimuth, coming from)
33 and 34	220 through 350
35	290 through 110
36	320 through 120
37 and 38	20 through 170
39	60 through 180

Significant wave height in the STWAVE model runs is reduced to account for depth-limited spectral breaking when waves are high enough and propagate into shallow enough water. Many of the design analysis stations are in sufficiently shallow water to be affected by depth-limited breaking. Although STWAVE accounts for the process, application of a transformation factor to offshore significant wave height may occasionally produce a height at nearshore stations that exceeds the realistic depth-limited maximum. To safeguard against unreasonably high significant wave heights at shallow water stations, heights are constrained to be at most 0.6 times the local water depth, including astronomical tide and storm surge. The value 0.6 is considered appropriate since limiting significant height over water depth ratios from STWAVE results of design concern range from around 0.55 to 0.62, with most values in the range of 0.55 to 0.58.

Maximum significant wave height by storm, needed to determine return period wave height values for structure design, is also extracted in Poplar_timehist.f along with corresponding peak period, wave direction, and water level. Separate output files are created for tropical storms only, northeasters only, and all storms together. For sta 2, these maximum values are shown for tropical storms and northeasters in Figures 35 and 36, respectively. Maximum values for sta 33, a west-facing station, are shown in Figures 37 and 38; maximum values for sta 36, a northeast-facing station, are shown in Figures 39 and 40; and maximum values for sta 39, a south-facing station, are shown in Figures 41 and 42. These values of maximum H_s for each storm, as well as associated peak period, direction, and depth, are tabulated for all stations of the northern expansion of Poplar Island in Appendix A.

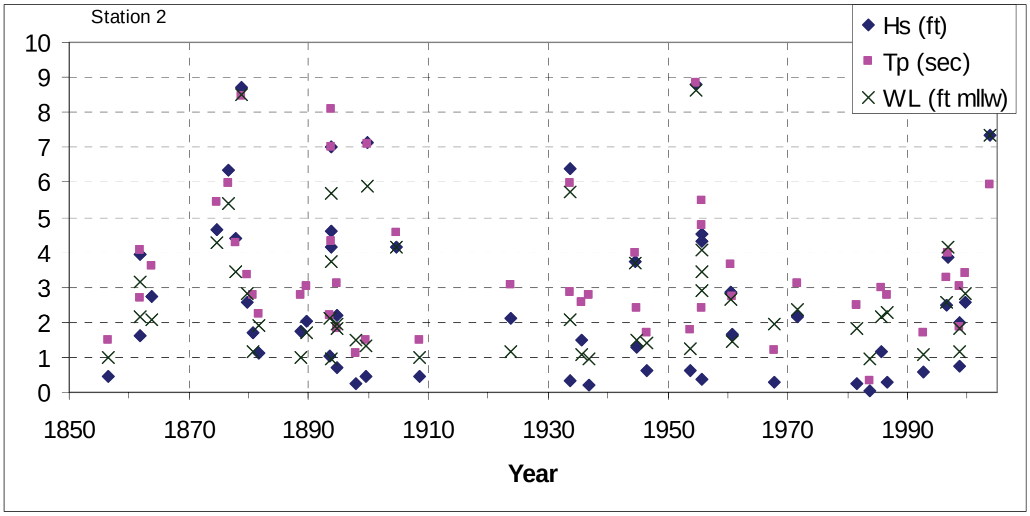


Figure 35. Maximum H_s and associated T_p and water level, Poplar Island, sta 2, tropical storms only

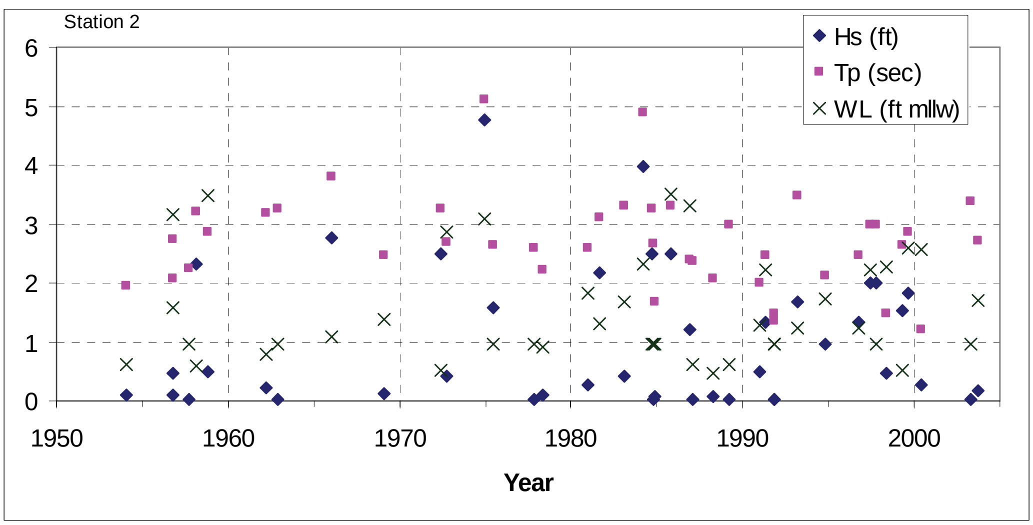


Figure 36. Maximum H_s and associated T_p and water level, Poplar Island, sta 2, northeasters only

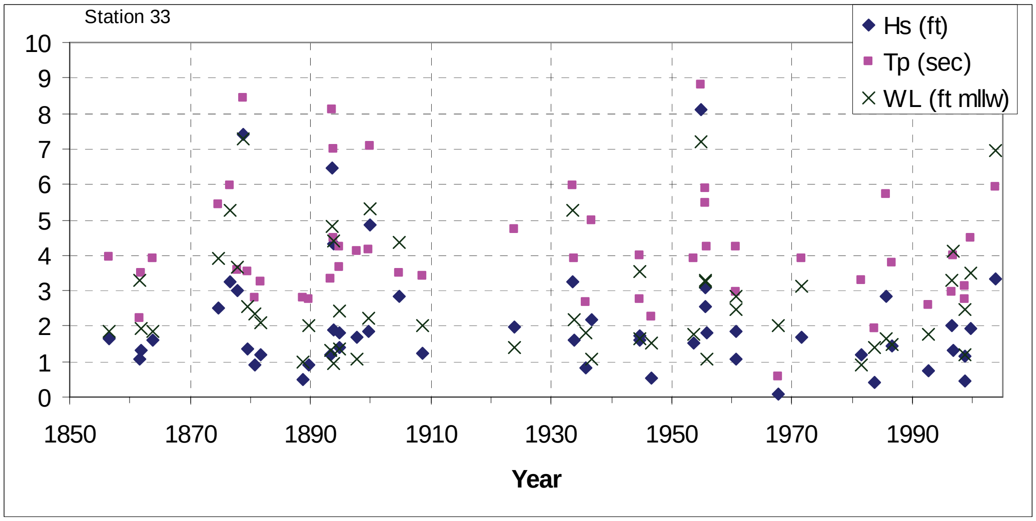


Figure 37. Maximum H_s and associated T_p and water level, Poplar Island, sta 33, tropical storms only

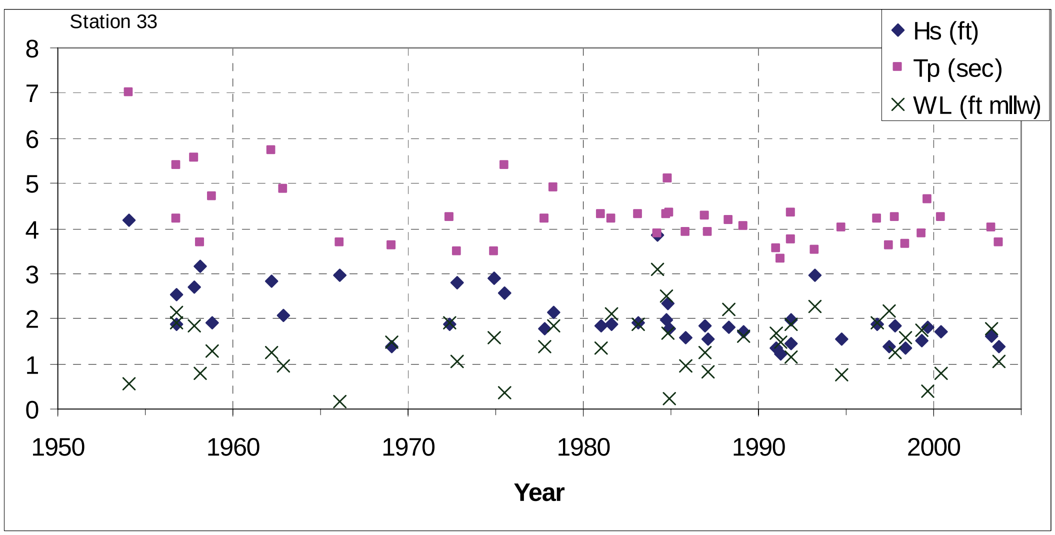


Figure 38. Maximum H_s and associated T_p and water level, Poplar Island, sta 33, northeasters only

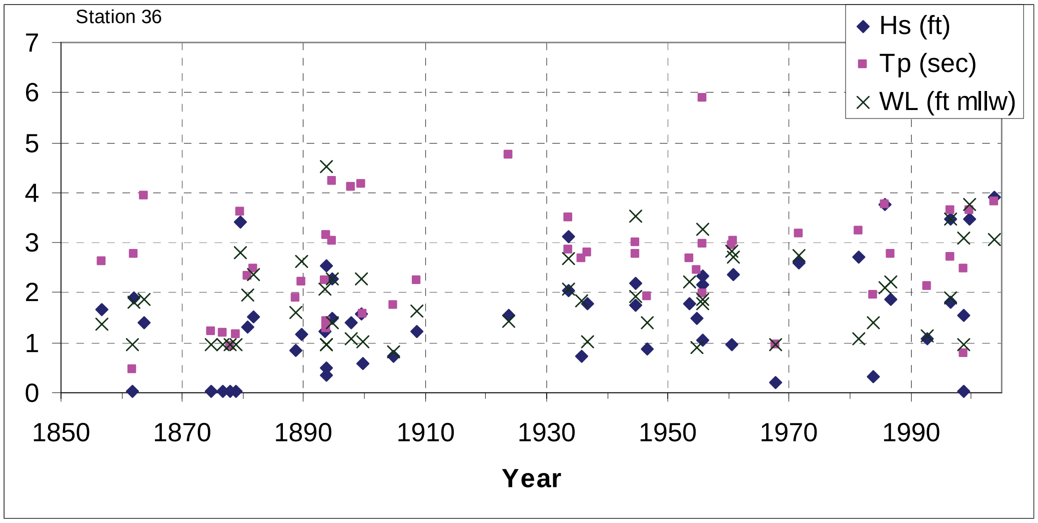


Figure 39. Maximum H_s and associated T_p and water level, Poplar Island, sta 36, tropical storms only

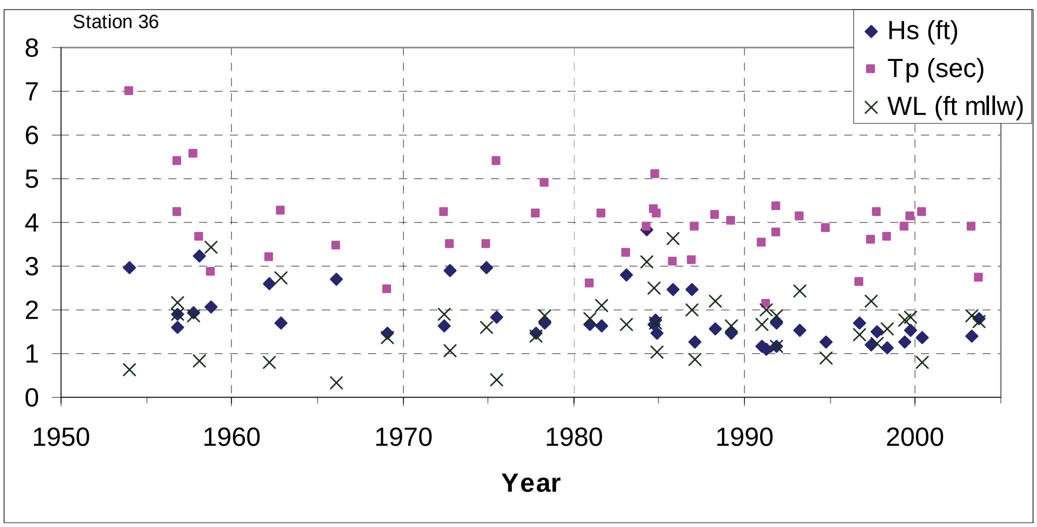


Figure 40. Maximum H_s and associated T_p and water level, Poplar Island, sta 36, northeasters only

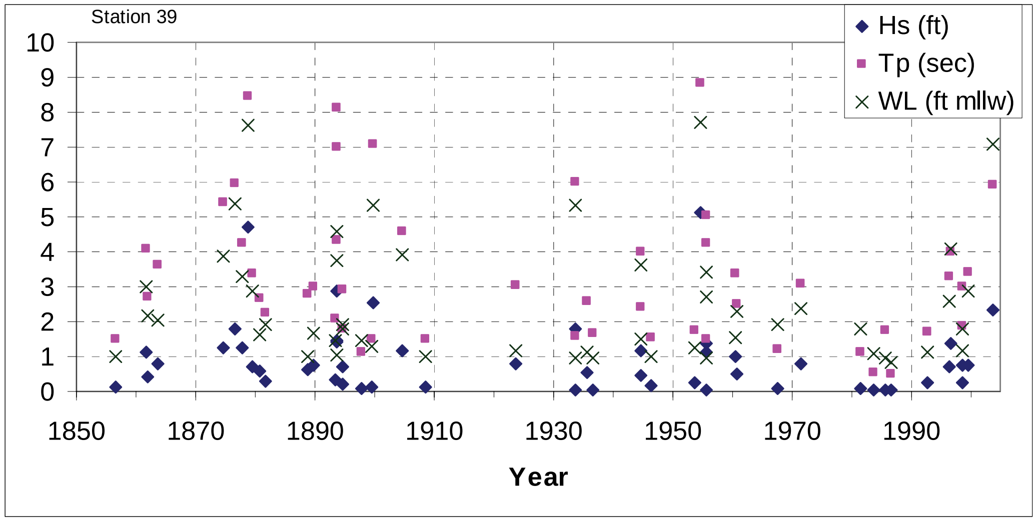


Figure 41. Maximum H_s and associated T_p and water level, Poplar Island, sta 39, tropical storms only

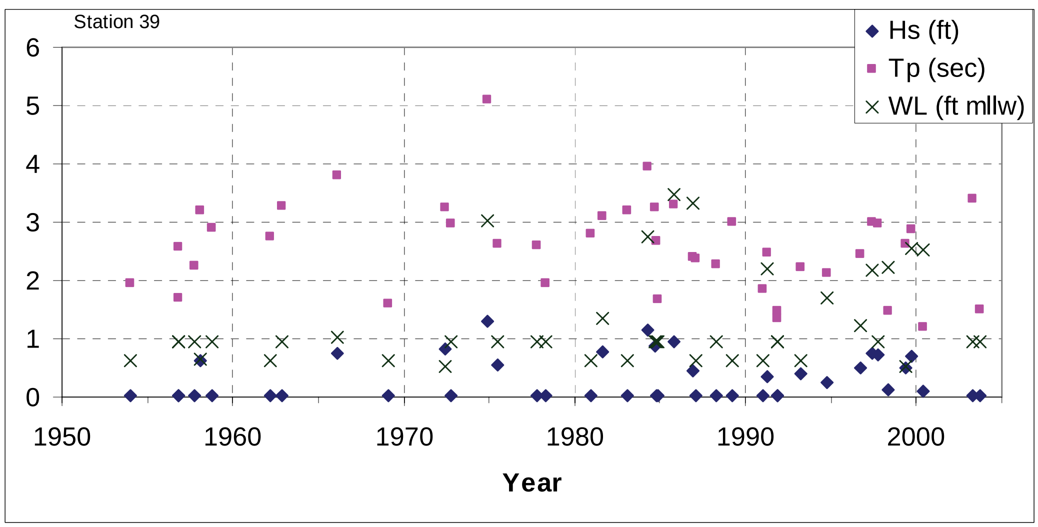


Figure 42. Maximum H_s and associated T_p and water level, Poplar Island, sta 39, northeasters only

Return period wave parameters

Maximum H_s values by storm at each station were subjected to extremal analysis using a modified version of the FORTRAN computer program that is the basis for the Extremal Significant Wave Height Analysis application in CEDAS/ACES. This program was used in preference to CEDAS/ACES so that multiple stations could be analyzed more easily and output information could be configured for convenient additional processing steps. As with CEDAS/ACES, the modified program follows the approach developed by Goda (1988). The modified program considers the same five candidate extremal distribution functions as in the CEDAS/ACES application and the distribution rejection and acceptance criteria proposed by Goda and Kobune (1990). In place of the CEDAS/ACES plot displays, the modified program produces a text file of plot information for the five extremal distribution functions for convenient display using a commercial spreadsheet program.

Extremal analysis of significant wave heights was applied to all storms together and to hurricanes only. Generally, H_s values at return periods of less than 30-50 years were dominated by northeasters and, for stations exposed to hurricane waves, H_s values at the longer return periods were dominated by hurricanes. Analysis of all storms included 179 storms over the 148-year time period. Analysis of hurricanes only included 52 storms over the 148-year period. The best-fitting extremal distribution was selected, based on the criteria of Goda and Kobune (1990) and a good visual fit to the return periods of concern for this project. Using the best-fit distribution, significant wave heights were determined for return periods of 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, and 100 years. For hurricane-influenced stations where the best-fit distribution for all storms underestimated H_s at the longest return periods, return period H_s was taken from the best fit for hurricanes only for return periods dominated by hurricanes.

To estimate an appropriate peak wave period and water level (also needed for structure design) to accompany each return period significant wave height, the computer program return_period_Tp.f is run. Inputs include return period significant wave heights and 148-year time history of waves and water levels at each station. The time history is screened to find all significant heights within a bin centered on the desired return period wave height. Bin widths considered are 0.2, 0.4, 0.6, 0.8, and 1.0 m (0.7, 1.3, 2.0, 2.6, and 3.3 ft). For example, the 50-year significant height at Poplar Island sta 2 is 2.24 m (7.34 ft). All cases in the 148-year sta 2 time history with significant height in the range of 2.14 to 2.34 m (0.2-m bin) [7.02 to 7.68 ft (0.7-ft bin)] were identified and their peak periods and water levels were averaged. The process was repeated for significant heights in the range of 2.04 to 2.44 m (0.4-m bin) [6.69 to 8.01 ft (1.3-ft bin)], 1.94 to 2.54 m (0.6-m bins) [6.36 to 8.33 ft (2.0-ft bin)], 1.84 to 2.64 m (0.8-m bins) [6.04 to 8.66 ft (2.6-ft bin)], and 1.74 to 2.74 m (1.0-m bin) [5.71 to 8.99 ft (3.3-ft bin)]. For each return period, a representative or *average* period and water level was chosen, with consideration of bins that captured enough cases to form a meaningful average but not so many cases as to dilute the target severe events. The extremal wave height analysis results for Poplar Island are tabulated and plotted in Appendix B.

Design Waves and Water Levels for James Island

The analysis of waves and water levels for James Island follows the same method used for Poplar Island. Figure 43 shows the design analysis station locations around James Island. Table 26 lists design water depths for James Island. Maximum significant wave height by storm, needed to determine return period wave height values for structure design, was extracted along with corresponding peak period, wave direction, and water level. As for Poplar Island, separate output files were created for tropical storms only, northeasters only, and all storms together. For sta 3, a southwest-facing station, these maximum values are shown for tropical storms and northeasters in Figures 44 and 45, respectively. Maximum values for sta 8, a northwest-facing station, are shown in Figures 46 and 47; maximum values for sta 11, a northeast-facing station, are shown in Figures 48 and 49. These values of maximum H_s for each storm as well as associated peak period, direction, and depth are tabulated for all stations of James Island in Appendix D. The extremal wave height analysis results for James Island are tabulated in Appendix E.

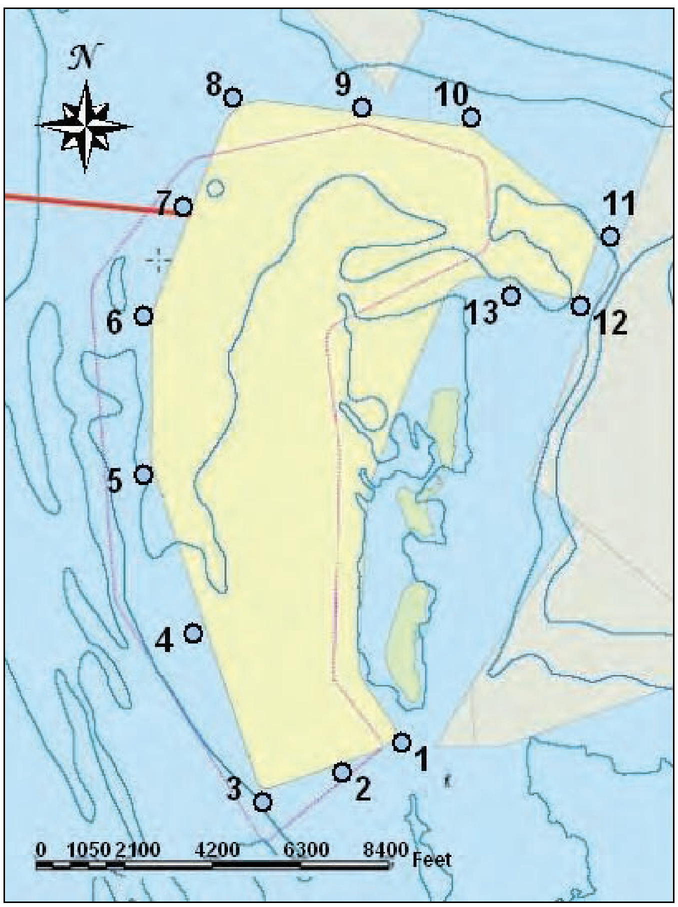


Figure 43. Design analysis stations, James Island

Table 26
Water Depths at James Island Design Analysis Stations

Station No.	Depth, m (ft) mllw
1	0.30 (0.98)
2	1.20 (3.94)
3	1.86 (6.10)
4	1.90 (6.23)
5	2.15 (7.05)
6	2.04 (6.69)
7	2.14 (7.02)
8	2.73 (8.96)
9	2.91 (9.55)
10	2.93 (9.61)
11	1.91 (6.27)
12	1.89 (6.20)
13	1.74 (5.71)

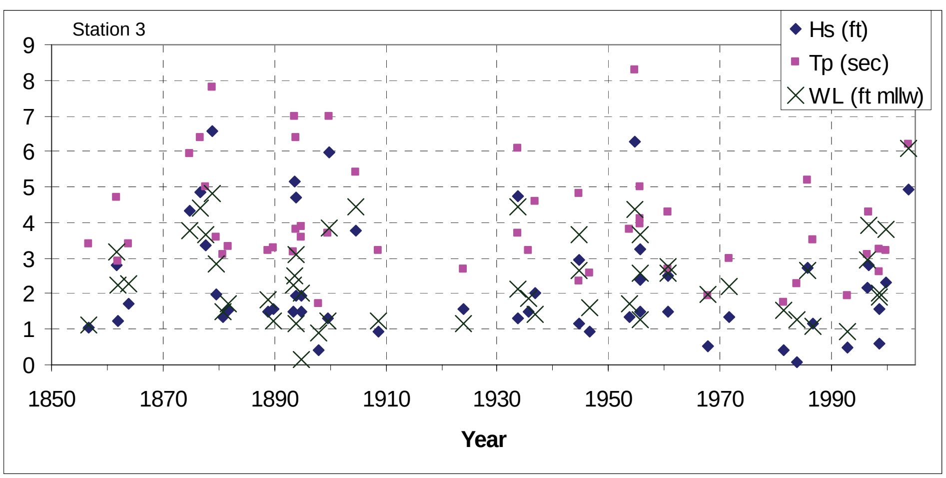


Figure 44. Maximum H_s and associated T_p and water level, James Island, sta 3, tropical storms only

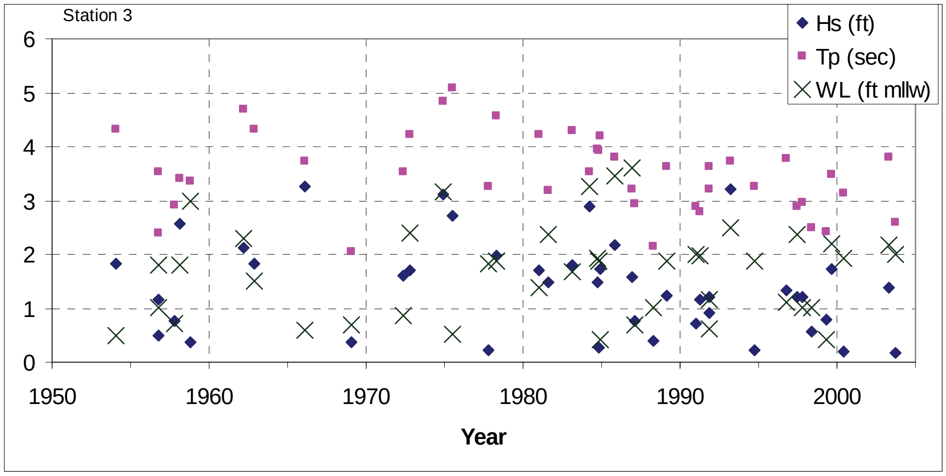


Figure 45. Maximum H_s and associated T_p and water level, James Island, sta 3, northeasters only

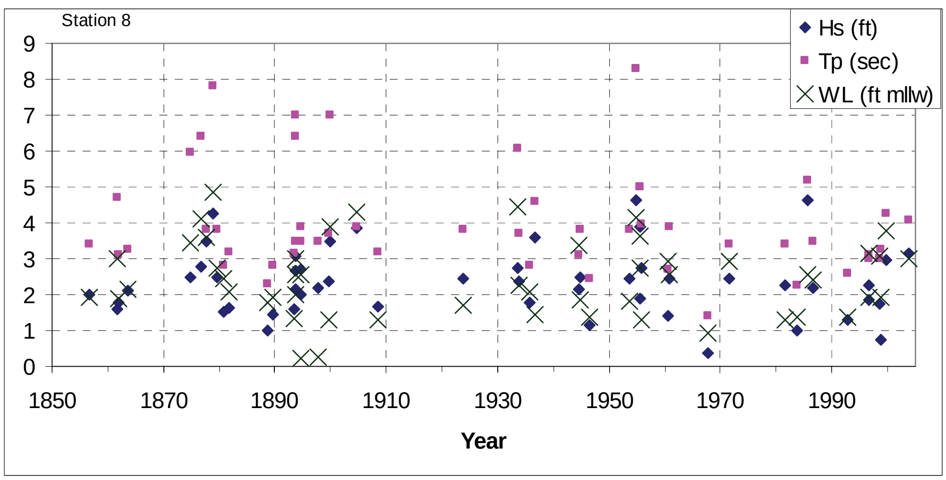


Figure 46. Maximum H_s and associated T_p and water level, James Island, sta 8, tropical storms only

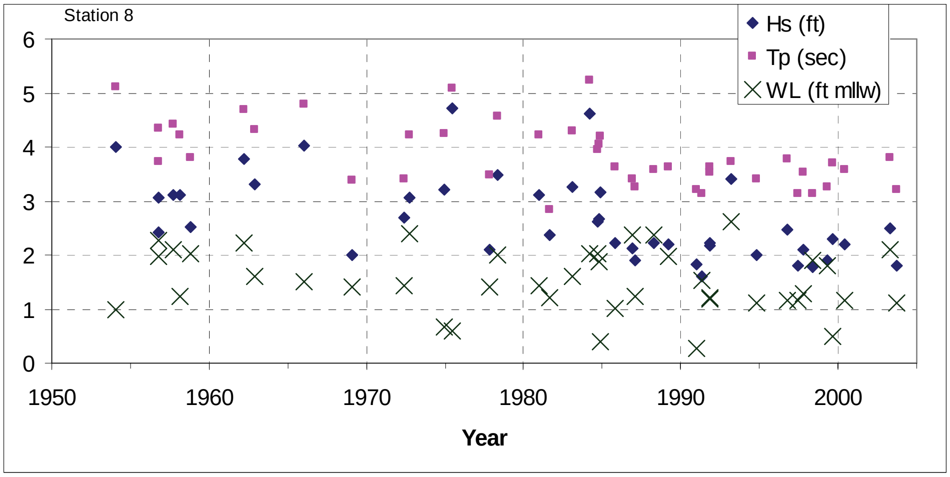


Figure 47. Maximum H_s and associated T_p and water level, James Island, sta 8, northeasters only

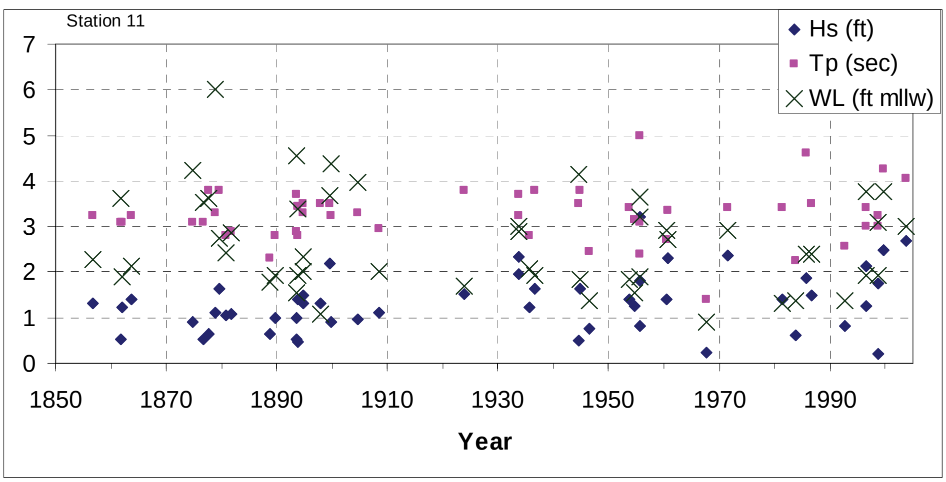


Figure 48. Maximum H_s and associated T_p and water level, James Island, sta 11, tropical storms only

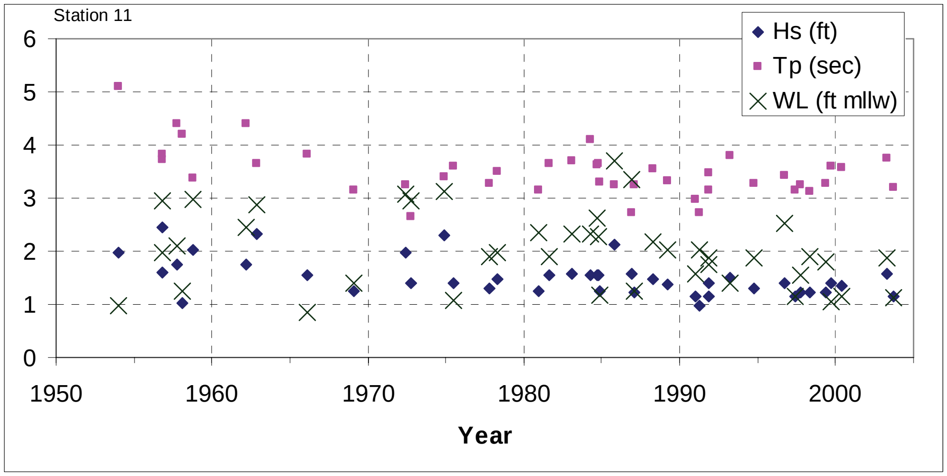


Figure 49. Maximum H_s and associated T_p and water level, James Island, sta 11, northeasters only

Design Waves and Water Levels for Barren Island

The analysis of waves and water levels for Barren Island follows the same method used for Poplar and James Islands. Figure 50 shows the design analysis station locations around Barren Island. Table 27 lists design water depths for Barren Island. Maximum significant wave height by storm, needed to determine return period wave height values for structure design, was extracted along with corresponding peak period, wave direction, and water level. As for Poplar and James Islands, separate output files were created for tropical storms only, northeasters only, and all storms together. For sta 3, a west-facing station at the center of the western leg of the structure, these maximum values are shown for tropical storms and northeasters in Figures 51 and 52, respectively. Maximum values for sta 5, a northwest-facing station, are shown in Figures 53 and 54; maximum values for sta 6, a north-facing station near the northern end of the structure, are shown in Figures 55 and 56. These values of maximum H_s for each storm as well as associated peak period, direction, and depth are tabulated for all stations of Barren Island in Appendix G. The extremal wave height analysis results for Barren Island are tabulated and plotted in Appendix H.

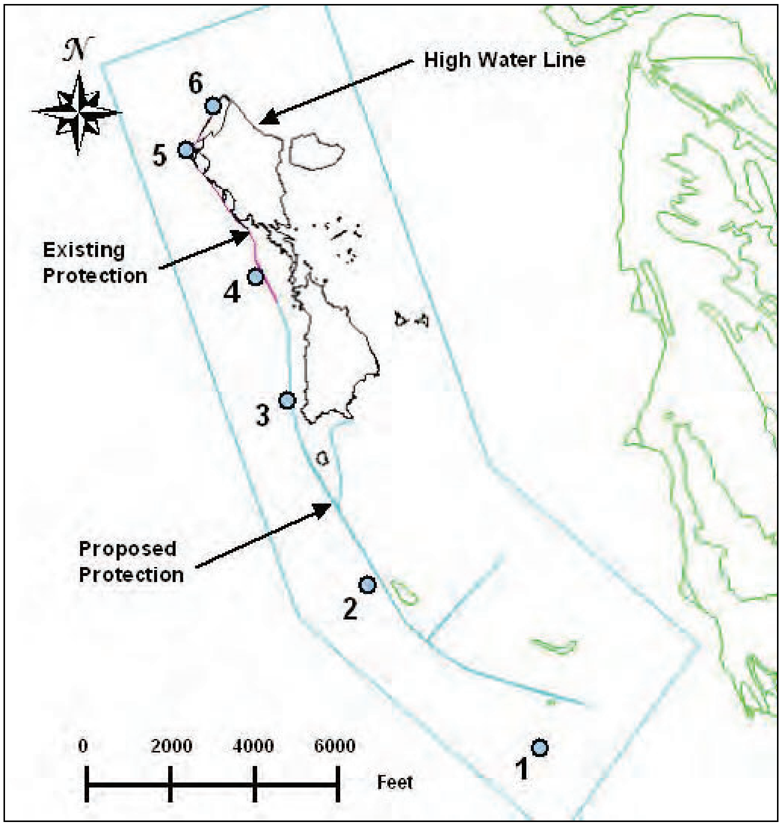


Figure 50. Design analysis stations, Barren Island

Table 27
Water Depths at Barren Island Design Analysis Stations

Station No.	Depth, m (ft) mllw
1	0.81 (2.66)
2	1.09 (3.58)
3	1.06 (3.48)
4	1.36 (4.46)
5	1.68 (5.51)
6	1.62 (5.31)

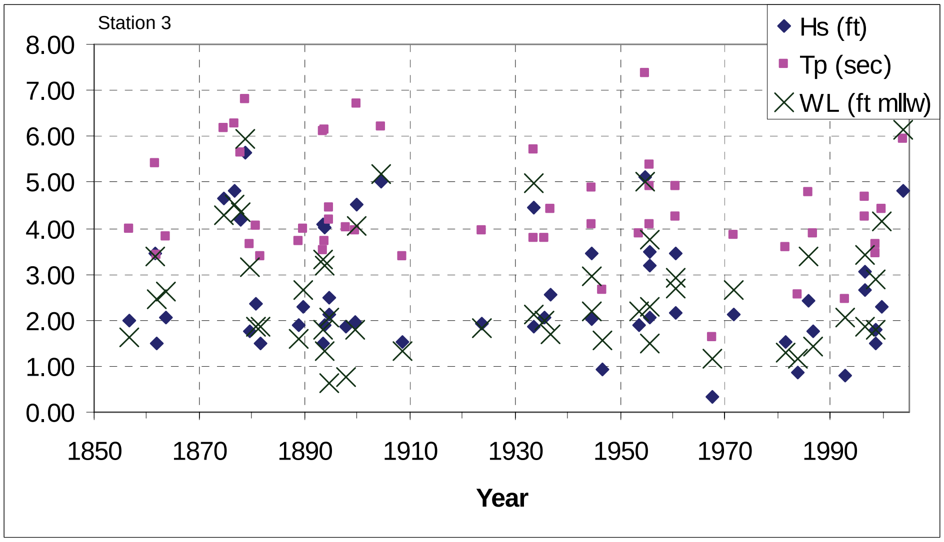


Figure 51. Maximum H_s and associated T_p and water level, Barren Island, sta 3, tropical storms only

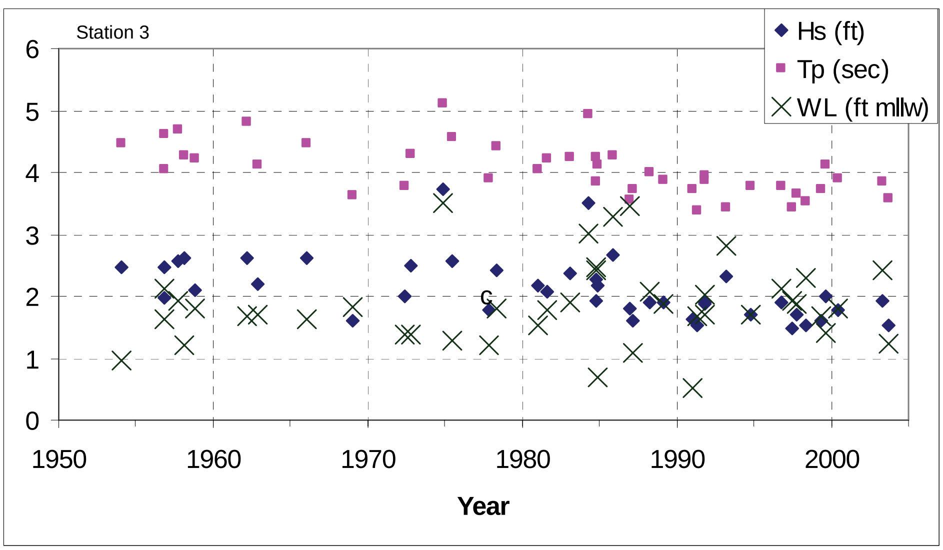


Figure 52. Maximum H_s and associated T_p and water level, Barren Island, sta 3, northeasters only

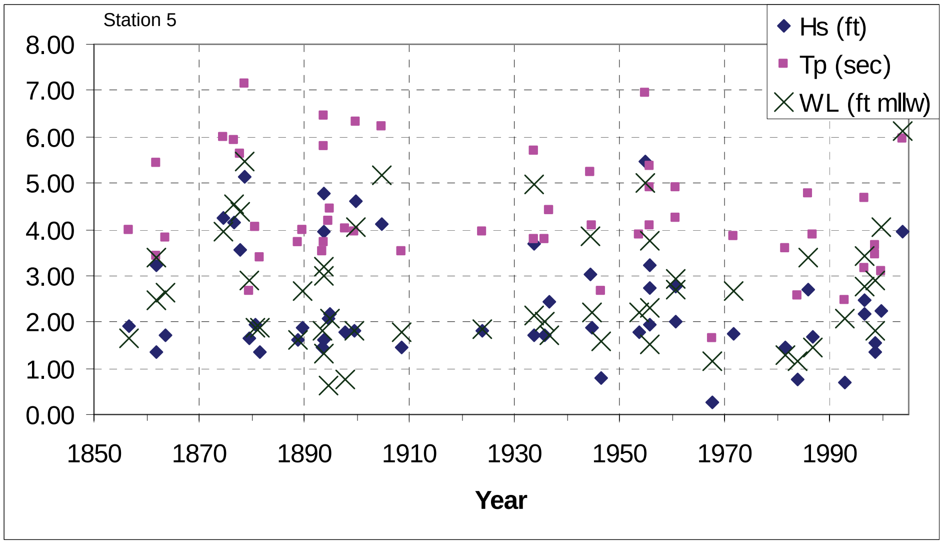


Figure 53. Maximum H_s and associated T_p and water level, Barren Island, sta 5, tropical storms only

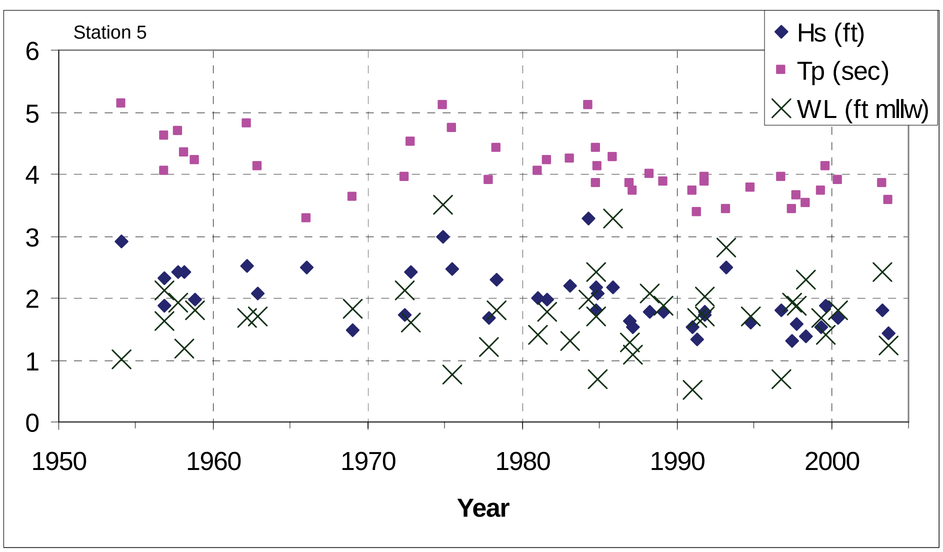


Figure 54. Maximum H_s and associated T_p and water level, Barren Island, sta 5, northeasters only

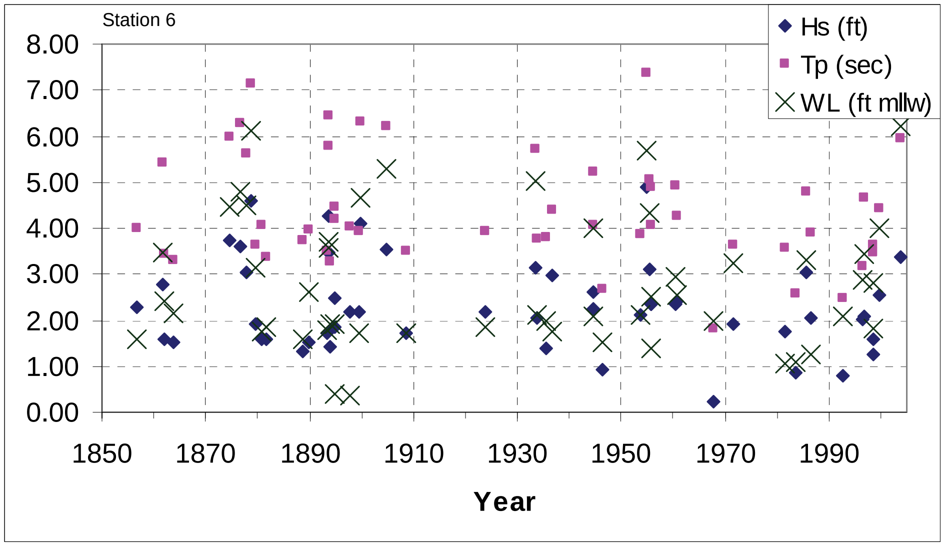


Figure 55. Maximum H_s and associated T_p and water level, Barren Island, sta 6, tropical storms onl

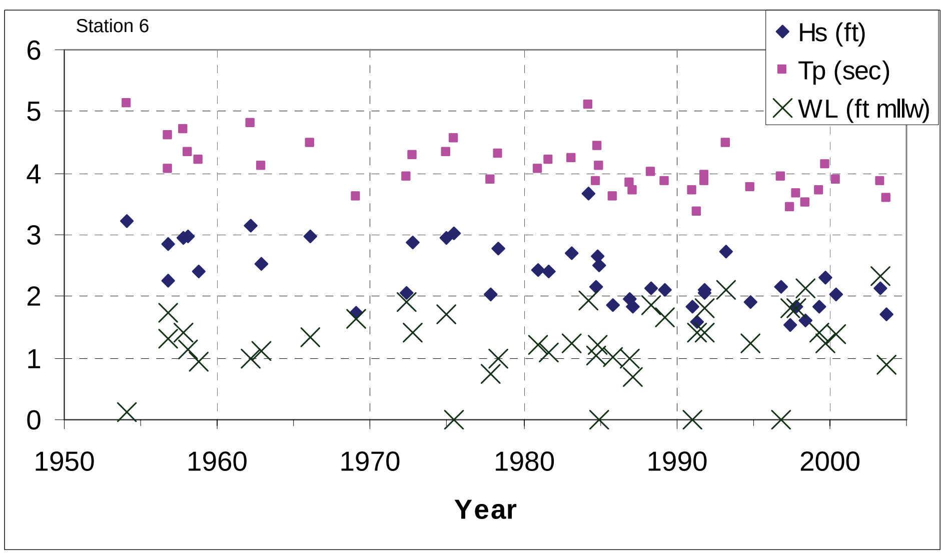


Figure 56. Maximum H_s and associated T_p and water level, Barren Island, sta 6, northeasters only

Simulation of Future Storm Sequences

Introduction

Future storm sequences are simulated with a multivariate time series Empirical Simulation Technique (EST) configured for life-cycle analysis. It has been named the Empirical Life-cycle Simulation (ELS) method to distinguish it from the traditional EST. The computer program for the wave and water level simulation portion of the ELS used in this study is called WELS and is a substantial extension and improvement of an earlier FORTRAN program described by Borgman and Scheffner (1991). The present program was developed in conjunction with this study and a similar study at Neah Bay, WA (Melby and Thompson 2005). The updated program is written in the much more advanced and graphics-friendly software, Matlab, extends the method to treat more than three time series at multiple locations simultaneously, introduces a more uniform way to handle month-to-month transitions, and includes various improvements in the methodology that have been developed in the 13 years since the writing of the earlier software. Although the new software version was developed specifically for this study and the Neah Bay study, it is expected to be easily adapted to other project applications.

Program overview

The WELS program consists of a suite of Matlab codes. The codes are organized into two parts:

a. **Analysis phase**. Empirical data time series are transformed to equivalent multivariate pseudo-Gaussian time series and all the basic arrays needed for the simulation process are computed and stored for later use.

b. **Simulation phase**. New pseudo-Gaussian time series are computed with frequency-domain Fast Fourier Transform (FFT) techniques, and then inversely transformed back to the empirical time series statistical structure.

These phases are illustrated in flowchart format in Figure 57. The components are discussed in additional detail in the following paragraphs.

Step 1: Data Preparation. The initial task is to convert the ASCII files of original wave and water level time series to Matlab data files in the *.mat format. The mat files are highly compressed and can be easily read in subsequent programs. The first step is to read the multiple historical wave and water level time series files described earlier in this chapter (one file per station) in ASCII format and organize the time series for all stations into a single large matrix. These files contain time series of waves and water levels near the toe of the structure. The next step is to load arrays created by the previous program and break up the massive matrix into smaller matrices, one for each oceanic property (significant wave height, peak wave period, wave direction, and water level) at each station.

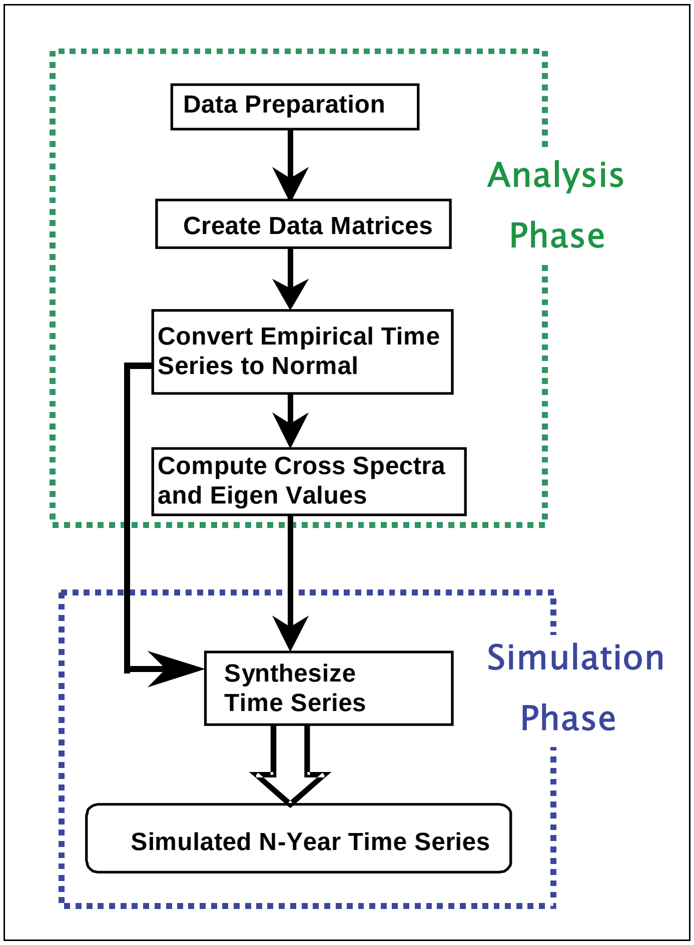


Figure 57. Flowchart of WELS wave and water level times series simulation for life-cycle analysis

Step 2: Convert Empirical Time Series to Normal. The first intrinsic WELS code unit computes empirical cumulative distribution functions and converts the empirical time series to pseudo-Gaussian time series. The basic steps in this task are to: (a) define the empirical cumulative distribution function (CDF) for each time series time step, (b) transform each data time series into an equivalent pseudo-Gaussian time series, and (c) save the inverse transform for use in the later simulation phase program.

Step 3: Compute Cross-Spectra and Eigen Values.

Step 3a: Cross-Spectral Estimation by Gaussian Smoothing. The cross-spectral analysis of the pseudo-Gaussian data time series proceeds through the following steps.

a. The time series are transformed to the frequency domain to obtain the complex-valued Fourier coefficients.

b. The cross-spectral estimates are obtained by smoothing the “raw” estimates.

c. The co- and quad-spectral densities are extracted from the real and imaginary parts.

Step 3b: Statistics of the Real and Imaginary Parts of the FFT Coefficients. If the pseudo-Gaussian time series are approximated as being truly multivariate normal, then a number of consequences for the real and imaginary parts of their complex Fourier coefficients result. The most important result is that the coefficients are independent of each other for $0 < m < N/2$ and simply related by complex conjugation to the coefficients of negative frequencies in $N/2 \leq m < N$. The implication of this is that the Fourier coefficients for each frequency, $m = 1, 2, 3, \ldots, m_0$, can be created separately without destroying the cross-correlations of the time series being simulated. The intercorrelation at a given frequency integer, m, can be forced into the simulation through the covariance matrix for the real and imaginary parts of the Fourier coefficients for the various wave properties being simulated. The covariance matrix depends on the spectra and the co- and quad-cross-spectra.

Step 3c: Simulation of Correlated Vectors with Eigenvectors. A multivariate normal column vector, $\boldsymbol{V}$, with mean zero and covariance matrix, C, can be simulated from a vector of independent normal random numbers, $\boldsymbol{Z}$ (of same size as $\boldsymbol{V}$), with the eigenvectors and eigenvalues of C. The columns of the orthogonal matrix G contain the eigenvectors, while L is a diagonal matrix whose elements on the diagonal are the corresponding eigenvalues. A simulation of $\boldsymbol{V}$ is given by:

$$V = G\, L^{1/2}\, Z \qquad (5)$$

The analysis phase of the WELS concludes with the computation of the matrix $GL^{1/2}$ from the covariance matrix for each m less than or equal to the cutoff, m_0.

Step 4: Synthesize Time Series. The synthesis phase of the WELS has two steps. First the pseudo-Gaussian time series for the wave properties are each simulated to generate the FFT coefficients, and then the coefficients are inverse-Fourier-transformed back to the time domain. The second step is to use the empirical CDF to reverse-transform the simulated pseudo-Gaussian time series back to the value scales of the original empirical data time series. The process is just the reverse of the steps in the analysis phase.

One step incorporated during the beta testing was remapping the wave height peaks and extreme water levels to better match the tails of the historical distributions. The stretching process was required because the extreme data points corresponding to extreme hurricanes, numbering less than 10, have unique statistical characteristics and were not well represented by the generalized process described above.

Comparison of simulation with original time series

Extensive plot comparisons of the simulated time series with the input data time series were done for quality control. Only three representative plots are shown in Chapter 7.

6 Life-Cycle Simulation Methodology – Structural Optimization

This chapter describes the procedures used for optimizing structure design within the framework of Empirical Life-cycle Simulation, or ELS. Procedures are described in terms of optimizing protective structures at the three islands: Poplar, James, and Barren. Methods discussed in the previous chapter were used to create suites of both historical 148-year as well as a possible future 50-year storm wave and water level life cycles that are statistically consistent with historical information. Candidate structure designs were then subjected to simulated and historical wave and water level life cycles, and the structure responses were analyzed. The optimization analysis described here has been developed using the latest rubble mound structure design guidance, presented in the *Coastal Engineering Manual* Part VI.

Overview

USACE planning policies and regulations for Civil Works water resource projects are stipulated in Engineer Regulation (ER) 1105-2-100, "Planning Guidance Notebook." The primary focus of this ER is to specify regulations required in planning projects that will produce the NED (National Economic Development) plan. The NED plan is the alternative plan with the greatest net economic benefit consistent with protecting the Nation's environment. Typically, the NED plan is the least expensive alternative over the projects economic lifetime, including first cost and maintenance costs as well as extraneous benefits and costs.

Structural optimization for the Poplar Island dike was described in Gahagan and Bryant et al. (1995). The details of the calculation methods described here are significantly different than those used in prior studies of the Poplar dike. However, there are many similarities with the methods for optimization described by Gahagan and Bryant et al. (1995). In this study, it is anticipated that the least-cost dike structure cross section that prevents breaching during the economic life will provide the structure portion of the NED alternative. Therefore, the basic objective of the optimization scheme described here is to minimize total amortized costs, including maintenance and first costs, with the constraint that breach failures over the economic life are to be avoided. These general objectives are the same as those described in Gahagan and Bryant et al. (1995).

The constraint of avoiding breaches is required to avoid large environmental costs associated with a breach-type failure. Breach failures can result in loss of sediment contained within the island. This sediment may spill out into surrounding areas and inundate oyster beds or areas of sensitive submerged aquatic vegetation (SAV). In addition, the sponsor suggested that a low-maintenance structure is crucial, as future maintenance funds are uncertain. First costs and maintenance costs vary depending on the design return period wave event. For shorter return period designs, the armor will be smaller and the crest height lower. A low initial cost design will produce higher maintenance costs and higher probability of breach failure. Designing for longer return periods produces a more reliable structure but costs more initially. The optimal design will be a balance between first costs and maintenance costs while avoiding structural failures.

Variables that are most influential in the optimization are crest height, armor stone size, and structure slope. Damage occurs primarily as a result of waves attacking and displacing armor stones and as a result of wave overtopping producing scour of the crest. Damage to the armor layer will progress in a predictable and continuous manner until the filter layers are exposed. At that point, the deterioration will accelerate until the structure is breached. There has been significant work recently on predicting damage development as a result of armor stone displacement (Melby and Kobayashi 1998a, 1998b, 1999, 2000). However, there has been little work on predicting the transition from significant damage to catastrophic breaching of the structure. Here, we conservatively assume that damage progresses very rapidly from exposure of the filter layers to breaching during one 3-hr increment. Damage to the crest due to overtopping is similar in that damage progresses slowly unless the overtopping exceeds a certain magnitude. At that point, the damage progresses very rapidly to a breached condition. Here, we assume that, for an unarmored crest, the structure progresses from minor damage to breach within one time step of 3 hrs if the overtopping rate exceeds this value.

Structure foundation failure also influences the design. Foundation failure is not evaluated in this report. However, input to this analysis from Gahagan and Bryant et al. (1995) and from the Baltimore District suggests that a seaward structure slope of 1V:3H or flatter is easier to construct and would be optimal from a geotechnical point of view. The Baltimore District has also suggested that portions of the structure may be able to be built with a 1V:2.5H seaward slope. Therefore, for this optimization analysis, slopes of 1V:2.5H and 1V:3H were evaluated for economic optimization. However, the majority of the analyses have been done with a slope of 1V:3H because the Baltimore District experience suggested that construction of the outer face of the sand core was much more difficult at steeper slopes.

Candidate structure designs were analyzed to determine an optimum design with FORTRAN computer programs written for this study. Within the programs, engineering response was computed primarily according to guidance published in the *Coastal Engineering Manual* (HQUSACE 2002). The engineering analysis was based on empirical equations given in the following sections. The step-by-step flow of the program is described in the final section of this chapter.

The following analysis is focused on the traditional multi-layer rubble mound revetment shown in Figure 58. This cross-sectional configuration is assumed for both Poplar and James Islands. One alternative for both Poplar and James Islands is the armored crest revetment shown in Figure 59. A third alternative includes building upland cells extending up from the initial lowland cells. This alternative would produce an upper slope to the revetment that would extend up from the landward side of the roadway on the low crest. Figure 60 shows the upland cell configuration. Barren Island includes some alternative sections, which will be described in more detail in the appropriate section.

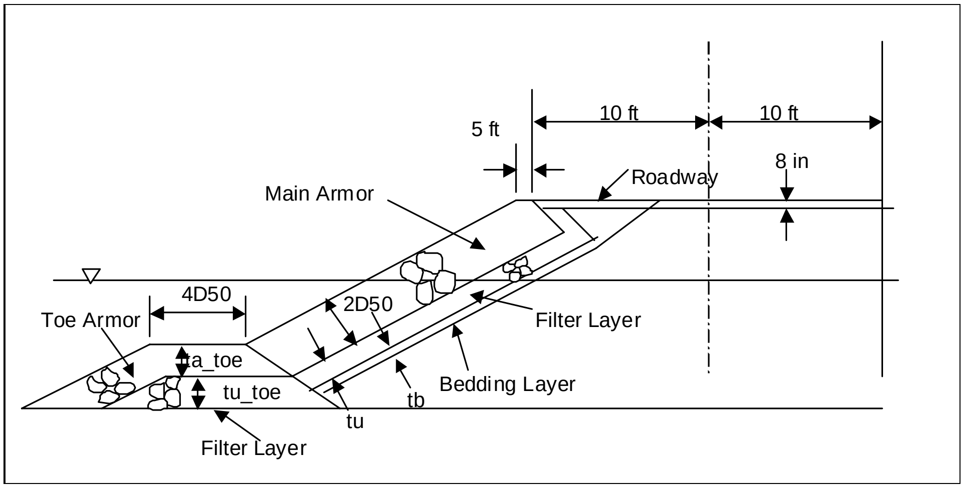


Figure 58. General revetment cross section

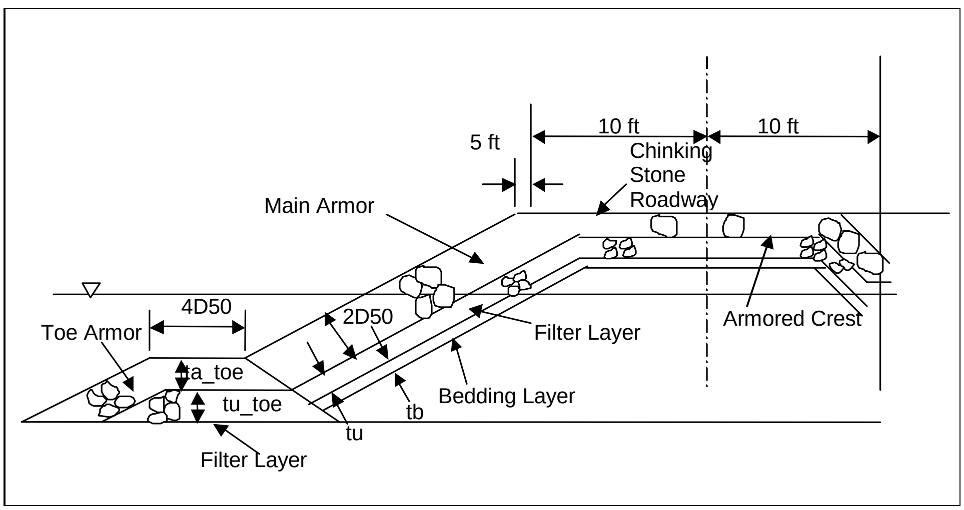


Figure 59. General cross section for armored crest alternative with single layer of armor across crest underlain by filter layer

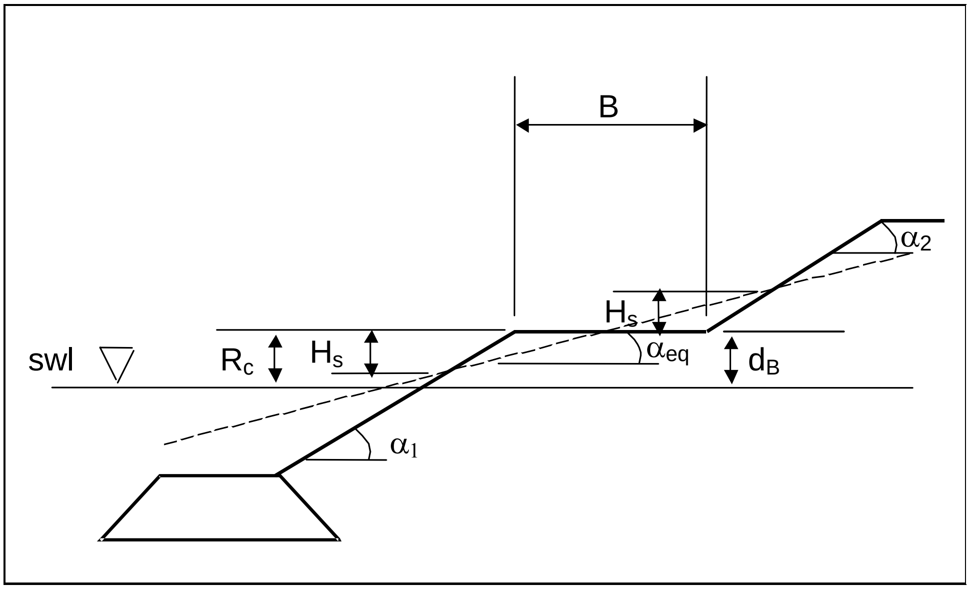


Figure 60. Definition sketch for runup on a compound slope

Wave Runup and Overtopping

Wave runup

Irregular wave runup on the structure is computed according to:

$$\frac{R_{2\%}}{H_s} = 0.96\xi_{om} \qquad 1.0 < \xi_{om} \le 1.5 \tag{6}$$

$$\frac{R_{2\%}}{H_s} = 1.17\xi_{om}^{0.46} \qquad \xi_{om} > 1.5 \tag{7}$$

$$\xi_{om} = \frac{\tan\alpha}{\sqrt{s_{om}}} \quad s_{om} = \frac{H_s}{L_{om}} \quad L_{om} = \frac{gT_m^2}{2\pi} \tag{8}$$

where

$R_{2\%}$ = wave runup height on the structure with 2 percent probability of exceedance

H_s = significant wave height, H_{mo} in this case, where $H_{mo} = 4(m_o)^{1/2}$ and m_o is the zero moment of the incident wave spectrum

α = structure seaward slope

T_m = mean wave period

g = acceleration of gravity

$\tan\alpha$ = structure slope from horizontal

L_{om} = deep water linear wave length based on the mean period

s_{om} = wave steepness based on the local wave height, deep water wave length, and mean period

ξ_{om} = Iribarren parameter based on the mean period

The *Coastal Engineering Manual* provides equations (Equations 9-17) for determining irregular wave runup on a compound slope. This technique is useful for determining the degree of runup on the upper slope of upland cells. Figure 60 shows a graphic idealization of a typical compound slope.

The runup relation for a compound slope is given by:

$$\frac{R_{2\%}}{H_s} = \begin{cases} 1.5\xi_{eq}\gamma_r\gamma_h\gamma_\beta & \text{for } 0.5 < \xi_{eq} \le 2 \\ 3.0\gamma_r\gamma_h\gamma_\beta & \text{for } \xi_{eq} > 2 \end{cases} \tag{9}$$

where the equivalent Iribarren number is given by Equations 10–17.

$$\xi_{eq} = \xi_{op}[1 - r_B(1 - r_{dB})] \qquad 0.6 \le \gamma_b \le 1.0 \tag{10}$$

$$r_B = 1 - \frac{\tan\alpha_{eq}}{\tan\bar{\alpha}} \tag{11}$$

$$r_{dB} = 0.5\left(\frac{d_B}{H_s}\right)^2 \qquad 0 \le r_{dB} \le 1 \tag{12}$$

$$\xi_{op} = \frac{\tan\alpha_1}{\sqrt{s_{op}}} \tag{13}$$

$$s_{op} = \frac{H_s}{L_{op}} \qquad L_{op} = \frac{gT_p^2}{2\pi} \tag{14}$$

$$\tan\alpha_{eq} = \frac{2H_s}{B + H_s(\cot\alpha_1 + \cot\alpha_2)} \tag{15}$$

$$\tan\bar{\alpha} = \frac{2H_s}{(H_s - d_B)\cot\alpha_1 + (H_s + d_B)\cot\alpha_2} \qquad -H_s \le d_B \le H_s \tag{16}$$

$$\begin{aligned} \tan\bar{\alpha} &= \tan\alpha_1 \quad \text{if} \quad d_B < -H_s \\ \tan\bar{\alpha} &= \tan\alpha_2 \quad \text{if} \quad d_B > H_s \end{aligned} \tag{17}$$

where

T_p = peak wave period corresponding to the peak frequency of the energy density spectrum

R_c = dike crest freeboard

α_1 = lower structure slope

α_2 = upper structure slope

γ_r = roughness correction = 0.55 (for two layers of rock armor)

γ_b = berm influence factor = ξ_{eq}/ξ_{op}

γ_h = depth-limited wave correction = 1.0 (must assume Rayleigh distributed waves without measurements)

γ_β = wave direction and directional spreading correction = 1.0 (for mostly head-on waves)

L_{op} = Airy wavelength based on the peak period

s_{op} = wave steepness based on the local wave height, deep water wave length, and peak period

ξ_{op} = Iribarren parameter based on the peak period

d_B = depth of berm crest, negative if the reference still-water level is below the berm crest

If $d_B < -H_s\sqrt{2}$ then $R_{u2\%} = R_c$. If the structure is breached, then the runup is not computed.

Wave overtopping

For an impermeable rough revetment, the volume rate of irregular wave overtopping per unit length of structure q is given by:

$$\frac{q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} = 0.06\exp\left(-5.2\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right) \tag{18}$$

$$\text{for}\ \ \xi_{op} < 2\ \ \text{and}\ \ 0.3 < \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{\tan\alpha}}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta} < 2 \tag{19}$$

and

$$\frac{q}{\sqrt{gH_s^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_s}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right) \tag{20}$$

for $\xi_{op} > 2$

According to the *Coastal Engineering Manual* Table VI-5-6, significant damage to a non-paved revetment crest will occur if $0.05 < q < 0.20$ cu m/sec/m ($4.0 < q < 16.1$ gal/sec/ft). In this study a lower limit of $q = 0.05$ cu m/sec/m (4.0 gal/sec/ft) was used for non-paved revetment crests. For paved or lightly armored crests, the lower limit value used was $q = 0.2$ cu m/sec/m (16.1 gal/sec/ft). For heavily armored crests, it was assumed that the main armor extended up over the crest. If the structure was breached, then the crest height was assumed to fall to the reference water level (mllw) and overtopping was computed for that crest height. The values of coefficients γ_r, γ_b, γ_h, γ_β were conservatively set as discussed in the preceding section.

Wave overtopping transmission

One alternative for the northern expansion of Poplar Island as well as the southernmost reach of Barren Island is a low-crested offshore structure. The primary functional criterion for design of this type of structure is overtopping transmission. Overtopping transmission $C_t = (H_{mo})_t/(H_{mo})_i$ is computed using the following relations:

$$C_t = \left(0.031\frac{H_s}{D_{n50}} - 0.24\right)\frac{R_c}{D_{n50}} + b \qquad 0.60 \le C_t \le 0.15 \tag{21}$$

$$b = -2.6s_{op} - 0.05\frac{H_s}{D_{n50}} + 0.85 \tag{22}$$

where $(H_{mo})_t$ is the transmitted significant wave height and $(H_{mo})_i = H_s$ is the incident significant wave height. The freeboard R_c is computed for overtopping using the return period water levels.

Armor Stability

Background

The Hudson equation is well known and has been used for years to determine armor stability. The equation in stability number form is given by:

$$N_s = \frac{H_s}{\Delta D_{n50}} = (K_D \cot\alpha)^{1/3} \tag{23}$$

where

Δ = $S_r - 1$

$S_r = \rho_r/\rho_w$ = specific gravity of stone

ρ_r = density of stone

ρ_w = density of water

$D_{n50} = (V_{50})^{1/3}$ = nominal stone diameter

$V_{50} = M_{50}/\rho_r$ = median volume of armor stone

M_{50} = median mass

K_D = empirical coefficient

α = structure seaward slope angle from horizontal

K_D takes into account all parameters not in the equation. The appropriate irregular wave height statistic has been discussed by many authors. Melby (2003) notes that recent guidance suggests H_s is reasonable if using K_D values published in the *Shore Protection Manual* (1984). The Hudson equation design assumes damage based on eroded volume of D% = 0 to 5.

As given in the *Coastal Engineering Manual*, van der Meer (1987) proposed equations in the late 1980s that are based solely on irregular wave experiments and explicitly include more parameters. These relations were given as:

For plunging waves where $\xi_{om} < \xi_{mc}$:

$$N_s = \frac{H_s}{\Delta D_{n50}} = 8.7P^{0.18}\left(\frac{S}{\sqrt{N_z}}\right)^{0.2}\xi_{om}^{-0.5} \tag{24}$$

For surging waves where $\xi_{om} > \xi_{mc}$:

$$N_s = \frac{H_s}{\Delta D_{n50}} = 1.4P^{-0.13}\left(\frac{S}{\sqrt{N_z}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_{om}^{P} \tag{25}$$

where

$$\xi_{mc} = [6.2P^{0.31}\sqrt{\tan\alpha}]^{\left(\frac{1}{P+0.5}\right)} \tag{26}$$

where

P = notional permeability

S = eroded area or damage

N_z = storm duration/T_m

Damage levels given by S = 1 to 3 represent the start of damage and correspond to Hudson's D% = 0 to 5 percent. For an impermeable dike, P = 0.1. For a traditional multi-layer breakwater, P = 0.4.

The wave height required in these equations is the significant H_s. Van der Meer suggested using two-percent exceedance value $H_{2\%}$ for application in shallow water. The stability equations for shallow water are identical except that the Rayleigh relation $H_{2\%} = 1.4H_s$ is substituted. The statistic $H_{2\%}$ must be determined from the actual measured time series of water surface elevation in shallow water. If $H_{2\%}$ is determined from a synthetic distribution, as it was for this study, a Rayleigh distribution must be assumed. In this case, the equations using $H_{2\%}$ are identical to those based on H_s. Further, Equations 24-26 were developed from primarily intermediate depth laboratory tests and do not explicitly incorporate depth. Therefore, Equations 24-26 are for primarily intermediate to deep-water applications and are not directly applicable to Poplar Island, where waves are mostly depth-limited.

Stability equations used in ELS

Melby and Hughes (2004) proposed stone stability equations specifically for both deep and shallow water applications. They derived stability equations based on the maximum momentum flux and fit to van der Meer's (1987) data. The fit was slightly better than that of Equations 24-26. They noted that the equations were based on first principles and would, therefore, be applicable to a wider range of conditions. In the following, the background for the maximum wave momentum flux is given.

Assuming irrotational potential flow on a locally flat bottom in water depth h, the wave-averaged and depth-integrated radiation stress is given by:

$$M = S_{xx} = \frac{1}{L}\int_{0}^{L}\int_{-h}^{\eta_x}(p_d + \rho_w u^2)\,dz\,dx \tag{27}$$

where

L = wave length

η_x = free surface location

p_d = dynamic pressure

ρ_w = fluid density

u = velocity in the x-direction

x = horizontal coordinate, an

z = vertical coordinate

The maximum depth-integrated wave momentum flux is given at the wave crest by:

$$(M_F)_{\max} = \int_{-h}^{\eta_x} (p_d + \rho u^2) dz \tag{28}$$

Using linear wave theory values for u and p_d yields:

$$\left(\frac{(M_F)_{\max}}{\rho_w g h^2} \right)_{\max} = \frac{1}{2} \frac{H}{h} \frac{\tanh kh}{kh} + \frac{1}{8} \left(\frac{H}{h} \right)^2 \left[1 + \frac{2kh}{\sinh 2kh} \right] \tag{29}$$

where

g = acceleration of gravity

H = wave height

k = $2\pi/L$ = wave number

In Equation 29, the first term on the right-hand side is the dynamic pressure term, while the second is the velocity term. In general, the pressure term will dominate. For example, for low steepness waves, the velocity term will only contribute 5 percent to the maximum momentum flux. For waves in shallow water at the steepness limit, the velocity term will provide the maximum contribution, roughly 30 percent of the momentum flux. Equation 29 assumes waves to be periodic and sinusoidal. But in shallow water, waves are nonlinear with peaked crests and shallow troughs. The wave forces from these nonlinear waves can be very different from those resulting from linear waves. Equation 29 will under-predict the momentum under the nonlinear wave crest.

The maximum wave momentum flux is highly nonlinear for nonlinear waves, such as steep waves in shallow water. This corresponds to the case where armor stability is at its minimum. It is desirable to develop a relation that can characterize the stability over the full range of water depths expected. Melby and Hughes (2004) described a non-linear wave momentum flux using a numerical Fourier solution. The resulting approximate relation was found to be:

$$\left(\frac{M_F}{\rho_w g h^2} \right)_{\max} = A_0 \left(\frac{h}{g T_m^2} \right)^{-A_1}$$

$$A_0 = 0.639 \left(\frac{H_s}{h} \right)^{2.026} \tag{30}$$

$$A_1 = 0.180 \left(\frac{H_s}{h} \right)^{-0.391}$$

The use of a nonlinear approximation for momentum flux is important because stability is at its minimum when the incident wave is the most nonlinear.

Two stability equations resulted from the fit to data using Equation 30. The recommended equations for stability were

For plunging waves:

$$N_m = 5.0(S / N_z^{0.5})^{0.2} P^{0.18} \sqrt{\cot \alpha} \quad s_m \geq s_{mc} \tag{31}$$

For surging waves:

$$N_m = 5.0(S / N_z^{0.5})^{0.2} P^{0.18} (\cot \alpha)^{0.5-P} s_m^{\;-P/3} \quad s_m < s_{mc} \tag{32}$$

where

$$s_{mc} = -0.0035 \cot \alpha + 0.03316 \tag{33}$$

and

$$N_m = \left(\frac{K_a (M_F / \gamma_w h^2)_{\max}}{(S_r - 1)} \right)^{1/2} \frac{h}{D_{n50}} \tag{34}$$

with $K_a = 1$ and $\gamma_w = \rho_w g$.

Equations 31 and 32 are analogous to Equations 24 and 25, and Equation 34 with Equation 30 is analogous to Equation 23. It is clear that the wave forcing portion given by Equations 30-34 provides a more rigorous prediction of the incident wave effect on stability. The inclusion of depth explicitly in Equation 34 through the maximum momentum flux is a significant improvement over Equations 24 and 25. The wave-structure interaction portion described by the right-hand side of Equations 31 and 32 is similar to that given in the van der Meer equations. Equations 30-34 are used here to predict zero-damage stone size for the toe layer as well as the armor layer for each return period wave condition at each station.

Accumulated damage

Although Equations 24-26 and 31-33 provide a way to predict damage on a structure, the damage is for constant wave conditions. The *Coastal Engineering Manual* provides equations to predict the normalized eroded cross-sectional area as a function of time for varying wave and water level conditions. The normalized eroded area as a function of time is given as:

$$\overline{S}(t) = \overline{S}(t_n) + a_p N_{mo}^{\,5} T_p^{-b} (t^b - t_n^b) \quad for \quad t_n \leq t \leq t_{n+1} \tag{35}$$

where

$\overline{S}(t_n) = A_e/D^2_{n50}$ = mean damage at time t_n

A_e = mean eroded cross-sectional area

$N_{mo} = H_{mo}/\Delta D_{n50}$ = stability number

a_p and b = empirical parameters

A similar equation uses time-domain wave parameters. The calibrated parameter values are $a_p = 0.022$ and $b = 0.25$. Note that S can be thought of as the number of stones displaced from a D_{n50}-wide cross section. The standard deviation of S was given as a function of the mean $\overline{S}(t_n)$ by the relation $\sigma_S = 0.5\,\overline{S}^{\,0.65}$. This standard deviation describes the alongshore variability of damage. Also given were relations for maximum depth of erosion, minimum remaining cover depth, and length of the eroded hole. The maximum eroded depth is d_e, the minimum remaining cover depth is d_c, and the maximum eroded length is l_e. These three parameters are normalized to obtain $E = d_e/D_{n50}$, $C = d_c/D_{n50}$, and $L = l_e/D_{n50}$. Melby and Kobayashi (1998a) expressed the key profile parameters as a function of the mean damage as follows: $\bar{E} = 0.46\,\overline{S}^{\,0.5}$, $\bar{C} = C_o - 0.1\,\overline{S}$, $\bar{L} = 0.44\,\overline{S}^{\,0.5}$ where C_o is the initial armor layer thickness.

A modification to Equation 35 was introduced by Melby and Kobayashi (1999) to allow for non-zero initial damage values. The modified equation is:

$$\begin{aligned} \overline{S}(t) &= 0.011 N_s^5 (N_e + \delta N)^{0.25} \\ N_e &= \left(\frac{\overline{S}(t_n)}{0.011 N_s^5} \right)^4 \\ \delta N &= (t - t_n)/T_m \quad for \quad t_n \le t \le t_{n+1} \end{aligned} \tag{36}$$

This equation is similar to Equation 35 but predicted damage is not dependent on the time that the simulation begins. In this report, accumulated mean eroded area is predicted using Equation 36 if the zero damage condition is exceeded at any point in the time series. The mean plus one standard deviation of damage is used for design. The parameters E, C, and L are also predicted if the zero-damage level is exceeded.

Toe stability

The toe berm for Poplar and James Islands can be either emergent or submerged, depending on the water level. The toe berm crest elevation was specified at +0.3 m (1 ft) mllw. This toe berm crest height was required in order to provide a quiescent area for construction of the sand-filled dike. Stability equations are given in the *Coastal Engineering Manual* (2002) for a submerged toe berm, for a low-crested stand-alone structure, and for a submerged stand-alone structure but not for a sometimes-emergent toe berm. The toe stability equation is only applicable to deep toes. There is no guidance directly applicable to the emergent toe berm shown in Figure 58. In this study, several equations are used to approximate toe stone size.

Deeply submerged toe. The stable toe berm stability number is given by:

$$N_s = \left(0.24\frac{h_b}{D_{n50}} + 1.6\right)N_{od}^{0.15} \tag{37}$$

or, rearranged, the nominal diameter is given by

$$D_{n50} = 0.625\frac{H_s}{\Delta N_{od}^{0.15}} - 0.15h_b \tag{38}$$

where

N_{od} = 1 for minimal damage

h_b = height of water surface above the berm crest = h - 1.0 ft - h_{mllw}

Equations 37 and 38 are limited to the condition where $0.4 < h_b / h_s < 0.9$.

Barely submerged toe berm. In the *Coastal Engineering Manual*, the submerged structure median stable weight W_{50} is determined using the following equations:

$$\frac{h_c^{'}}{h} = (2.1 + 0.1S)\exp(-0.14N_s^*) \tag{39}$$

or, after rearranging,

$$N_s^* = -\left(\frac{1}{0.14}\right)\ln\left(\frac{h_c^{'}}{h}\frac{1}{(2.1 + 0.1S)}\right) \tag{40}$$

$$W_{50} = \gamma_r D_{n50}^3 = \gamma_r \left(\frac{H_s s_p^{-1/3}}{\Delta N_s^*}\right)^3 \tag{41}$$

$$s_p = \frac{H_s}{L_p} \quad \Delta = S_r - 1 = \frac{\rho_r}{\rho_w} - 1 \tag{42}$$

where

h'_c = height of the toe berm above the bottom

h = water depth seaward of the toe berm

N_s* = $H_s/(\Delta\, D_{n50}\, s_p^{-1/3})$ = spectral stability number

L_p = wave length corresponding to the peak spectral frequency at the toe of the structure

W_{50} = median stone weight

γ_r = stone specific weight

Low-crested toe berm and low-crested stand-alone mound. The stable weight for a rubble mound structure was discussed earlier in the chapter. Equations 31-34 assume a traditional two-stone-thick armor layer and filter layers below the armor, as well as a stable toe. These equations also assume little or no overtopping. The stable weights from Equations 31–34 can be modified for the condition of a heavily overtopped low structure crest. The low-crest stability modification suggested by the *Coastal Engineering Manual* in Table VI-5-24 was used in the ELS analysis. The modification reduces the stable armor weight by a small amount, the amount increasing as the crest approaches the still water level. The reduction relation is given by:

$$f_i = \left(1.25 - 4.8\frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}\right)^{-1} \quad 0.8 \le f_i \le 1.0$$
$$0 < \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}} < 0.052 \tag{43}$$

and is applied to the nominal median diameter. So, in the stability equations, D_{n50} is replaced by $f_i D_{n50}$. The equation above is limited to $R_c > 0$.

Equation 43 is for emergent structures only. This method results in a more conservative stone size than if a submerged armor weight stability equation is used. For the condition where the water level is at the structure crest, $f_i = 0.8$ and the reduction in D_{n50} is 20 percent. As the freeboard increase to roughly 1 m (3 ft), f_i approaches 1.0, depending on the wave height and wave period.

Computing toe berm stone size. The water depth has significant influence on the toe berm armor size. A low water level with a low-crested emergent toe berm will usually demand a larger armor stone than a submerged toe berm. In the ELS calculations, the toe berm armor size was computed for a range of water depths from the highest water level for each return period down to the mllw level. The program LC_COST_REV includes a small loop for computing the stable armor size for seven depths from high to low water using the appropriate toe stability equation: Equation 38 for $h_b/h_i \ge 0.3$, Equations 39-42 for $0.0 < h_b/h_i < 0.3$, or Equation 43 for $R_c \ge 0$, where h_i is the depth at each increment of the loop and $i = 1$ - 7. If appropriate, as the water level was reduced, the wave height was reduced to the breaking limit according to the relation $H_b = 0.6h_i$. The breaker height index of 0.6 was appropriate for this site, as described in Chapter 5. The stable toe stone size was selected as the maximum of the seven computed for each return period. Note that Equation 37 with 38 does not always converge. If the equation did not converge, the wave height was adjusted upward until the equation converged.

As stated earlier, the southern extension to the Barren Island structure was proposed to be an offshore low-crested rubble mound breakwater. Equation 43 was applied to this structure in order to size the armor stone.

Economic Present Worth

The relation to determine the present worth of first cost and future maintenance costs is:

$$PW_m = C_m \left[\frac{1+i}{1+p} \right]^{N_m} \tag{44}$$

where

PW_m	=	present worth of cost C_m
C_m	=	cost *m* in today's dollars
I	=	inflation rate = 0.03
p	=	prime interest rate = 0.05375
N_m	=	number of years between today and the date of cost C_m

Corps policy for determining the NED alternative dictates that the inflation rate is zero so that benefits are not inflated. However, this minimizes the importance of repairs in out years. For the Chesapeake Bay islands, infrequent hurricanes can cause significant damage, including structure breaching. If these major repairs are required in the latter half of the economic life, they will have a negligible contribution to the total present worth cost if the inflation is assumed to be zero. Baltimore District engineers suggested that they would like to minimize the potential for these large breaches and associated repairs because a breach will result in sediment contamination of important ecological areas (e.g., oyster beds and SAV areas). As such, it was determined that an inflation rate should be used in the simulations. The inflation rate was set at 0.03. Cleanup cost for sediment contamination is not incorporated in this analysis. However, it is expected that environmental cleanup would increase the repair costs for a breach significantly.

The total present worth PW_T is computed by simply summing the PW_m values for all costs during the project's life cycle.

$$PW_T = \sum_{m=1}^{N} PW_m \tag{45}$$

The present worth total cost can be annualized using the relation

$$A_T = PW_T \left[\frac{r(1+r)^N}{(1+r)^N - 1} \right] \tag{46}$$

where r = the annualizing interest rate (0.05375) and N = the economic design life of structure (50 years).

A cost relation was derived to account for the fact that the structure length of a constant cross section will typically be significantly longer than the repair length. The final total cost for a section of structure of constant cross section is given by:

$$\begin{aligned} PW_T / L_s = & \left[\sum FMC + \sum \frac{FFC}{L_s} \right] \left(\frac{1}{1+p} \right)^{lag} + \\ & \frac{L_r}{L_s} \sum \left[\left(RMC + \frac{RFC}{L_r} \right) \left(\frac{1}{1+p} \right)^{N_{mr}} \right] \end{aligned} \tag{47}$$

where

L_s	=	length of structure of constant cross section
L_r	=	length of repair
lag	=	time until initial construction
FMC	=	initial construction material cost per unit length of structure (e.g., armor layer cost for initial construction)
FFC/L_s	=	initial construction fixed cost per unit length of structure (e.g., mobilization cost)
RMC	=	repair material cost per unit length of structure
RFC/L_r	=	repair fixed cost per unit length of repair

PW_T was computed for several ratios of L_r/L_s. The length of sections between design analysis stations on Poplar Island is roughly 244 to 1463 m (800 to 4,000 ft). The repair length for breaches and associated repairs on Poplar Island from Hurricane Isabel was roughly 122 m (400 ft). Therefore, L_r/L_s = 0.1 - 0.5. It is assumed that other islands would be similar. For this study, L_r/L_s values used were 0.1, 0.3, and 0.5. A value of 0.3 was used for most calculations as an average. The present worth per unit structure length resulting from Equation 47 can be multiplied by L_s for each section to get the total present-worth cost for that section. Ranges for FMC, FFC/L_s, RMC, and RFC/L_r were developed based on the initial construction costs for Poplar Island and the Hurricane Isabel repair of the Phase II southern dike.

Structure Life-Cycle Analysis Program

The ELS structure analysis process developed for this study is summarized in more detail in this section.

a. Wave and Water Levels.

(1) Time series. Chronological historical wave height, wave period, wave direction, and water levels at a number of stations near the toe of the structure at 3-hr intervals for 148 years were placed in a file named *wavefile.txt*. Water levels were referenced to mllw.

(2) Extremal distributions. Given historical waves and water levels at the toe of the structure, the long-term distributions of maximum storm significant wave heights were determined. From those distributions, wave heights and corresponding wave periods and water levels were chosen for representative return periods ranging from 5 to 100 years.

(3) Time series simulation. The historical wave and water level climate was simulated using the ELS method described in Chapter 5. For this study, 50 simulations were generated for each design analysis station spanning 50 years each.

b. Structural Analysis.

(1) Prepare input data. Analysis constants, material descriptions, and material costs were assembled and stored in program input file

Damage-Input.txt. There is a unique input file for each design analysis station. The computed extremal wave height, wave period, and water level for each return period were also placed in this input file. Table 28 lists input file parameters that were assigned constant values in the simulations. Table 29 lists input file parameters that varied among the simulations and gives the range of values simulated.

(2) Run FORTRAN program LC_COST_REV. Structural analysis FORTRAN program LC_COST_REV reads *wavefile.txt* and *Damage-Input.txt*. The program computes a representative zero-damage cross-section for each return period wave and water level condition in the input file. Values computed include:

(a) Armor weights. Primary median armor weight, W_{a50}, was based on stability Equations 30-34, and toe median armor weight, W_{ta50}, was based on stability Equations 37-42. For a low-crested structure, armor size is modified according to Equation 43.

(b) Filter layer and core material sizes. Filter layer median weight was $W_{u50} = W_{a50}/10$ for primary armor and $W_{tu50} = W_{ta50}/10$ for toe armor. The bedding material was assumed to be quarry-run material. The road surface was assumed to be graded gravel. The core of the dike structures was assumed to be sand.

(c) Armor and filter layer thicknesses. The armor layer thickness was computed as $t_a = 2D_{n50}$, while the filter layer thickness was computed as $t_u = 2D_{u50}$, where $D_{u50} = (W_{u50}/\gamma_r)^{1/3}$.

(d) Cross-sectional area and total weight of each material for the given cross section.

(e) Initial cost of cross section. This cost was used for both initial cost and breach repair cost. Any sand fill is not included in this calculation.

For each return period and corresponding cross-sectional design, the program steps through the wave and water level time series file, computing the zero-damage stability number and the actual stability number based on Equations 30-34 at each time step. If the zero-damage stability number is exceeded, then damage is computed and accumulated using Equation 36. Toe damage is computed at each time step according to Equations 37-42. Toe damage is not accumulated because the damage in Equations 37-42 is for single events. There are presently no toe damage accumulation relations. If damage occurs, the details of the damage are output to a file. If primary armor damage exceeds either the minor damage or the breach damage limits, the repair flag is set and a time counter is started. The structure is repaired to its original condition if the mobilization time limits for minor repair or breach repair are exceeded, depending on the level of damage. At that point, all counters and damage levels are reset to zero. If a repair occurs, the details are output to a data file. The damage limits as well as the repair time limits are inputs in file *Damage_Input.txt*. Note that if minor damage is caused by a storm, greater damage or even a breach could result on the damaged structure before the repair mobilization time is completed. If a breach occurs on an already damaged structure, the repair counter is restarted, as it is assumed that mobilization and funding for breach repair are significantly different from mobilization for minor repair.

Wave runup and wave overtopping are computed at each time step. Runup is computed using Equations 6-8, runup on a compound slope using Equations 9-17, and overtopping volume using Equations 18-20. If wave overtopping exceeds the predefined damage limits, the section is assumed to be breached. In this case, the breach time counter is started. The structure is repaired to its original condition if the time limits for breach repair are exceeded. If the structure is low-crested, overtopping transmission is computed using Equations 21 and 22.

Present-worth costs per unit length are computed for initial cost and for each repair that is instigated using Equations 44-47. Note that there is no cost associated with damage unless a repair occurs. Present-worth costs are accumulated throughout the life cycle.

To summarize the output files described previously, output from the program includes summaries of all damage, overtopping, and repair, as well as overall summaries. Damage is output for the sporadically repaired structure and for the case if no repairs were done. An economic detail file of all repairs and an economic summary file are also output. A time history of damage, runup, and overtopping is written. Initial material volumes and material costs are also output for each return-period cross section in summary files. Cumulative damage, repairs, and costs are also output for each event as they occur.

Table 28
Fixed Parameter Values for Structure Analysis

Parameter	Variable	Value
Permeability	P	0.1
Porosity	Por	0.38
Stone specific gravity	S_r	2.578
Stone density	ρ_r	2.644 tonnes/cu m (165 pcf)
Minor repair limit	S_M	8
Breach repair limit	S_B	18
Minor repair time limit	-	180 days
Breach repair time limit	–	120 days
Roughness parameter	γ_b	0.55
Crest width	B	7.62 m (25 ft)
Upper structure slope	α_2	1V:3H
Toe berm height	d_B	+0.305 m (1 ft) mllw
Toe berm seaward slope	cot φ	2
Toe berm leeward slope	cot β	1.5
Toe berm crest width	-	$4D_{toe}$
Toe armor thickness	-	$2D_{toe}$
Allowable main armor damage	S	1.0
Allowable toe damage	N_{od}	1.0
Number of waves for zero damage	N_z	7000
Inflation or escalation rate	i	0.03 or 0.0
Interest rate	R	0.05375
Economic life	N	50 years
Armor material unit cost	-	$56/tonne ($50.4/ton)
Filter material unit cost	-	$39/tonne ($35.1/ton)
Bedding material unit cost	-	$44/tonne ($39.6/ton)
Quarry-run material unit cost	-	$44/tonne ($39.6/ton)
Geotechnical material unit cost	-	$4.78/sq m ($0.44/sq ft)
Lag before initial construction	Lag	2

Table 29
Parameter Ranges for Structure Analysis

Parameter	Variable	Values
Overtopping limit	q_{allow}	0.05, 0.20 cu m/sec/m (4.0 gal/sec/ft, 16.1 gal/sec/ft)
Structure crest height	R_c	2.44 – 4.27 m in increments of 0.15 m (8.0 - 14.0 ft in increments of 0.5 ft)
Fixed first cost	FFC/L_s	\$500/m, \$1000/m (\$152/ft, \$305/ft)
Fixed repair cost	RFC/L_r	\$2,500/m, \$1,000/m (\$762/ft, \$305/ft)
Structure slopes	Cot α	2.5, 3.0, 3.5, 4.0
Ratio of repair length to section length	L_r/L_s	0.1, 0.3, 0.5

7 Life-Cycle Simulation Results, Poplar Island

This chapter describes the life-cycle structural optimization of the north extension (Phase III) of Poplar Island. Wave and water level results are presented in the following section. Methods used to develop these results are discussed in Chapter 5. Structure response and optimization are presented in the second section of this chapter. The methodology used to optimize the design of Poplar Island protective structures is given in Chapter 6.

Waves and Water Levels

The extremal H_s values for various return periods at each Poplar Island station are shown in Figure 61. The results are tabulated and plotted independently for each station in Appendix B in order to provide more background information. Stations with an open exposure toward the south experience the highest waves. These are also the stations most dominated by hurricanes. North- and east-facing stations along the north end of Poplar Island are less dominated by hurricanes. Return period H_s is relatively low at these stations, and the difference in H_s between the shortest and longest return periods is relatively small. For example, of the stations around the proposed extension, three have some exposure toward the south. Stations 33 and 34 face west but are also open to the south-southwest. Station 39, on the relatively protected back side of the island, also has some exposure toward the south, though it is partially obstructed by Poplar and Coaches Islands. Return period H_s at sta 33 through 39 follow a smooth variation around the proposed expansion for return periods up to about 40-50 years. For longer return periods, the impact of hurricanes causes a noticeable increase in H_s at sta 33, 34, and 39 relative to the other stations. Return period values of peak wave period and water level from this analysis are illustrated in Figures 62 and 63.

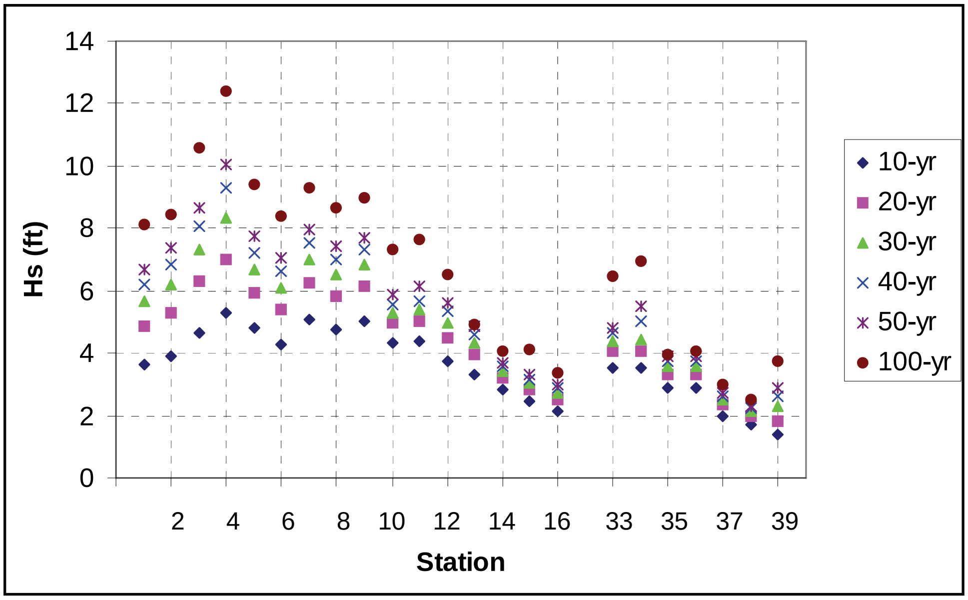


Figure 61. Return period H_s at nearshore stations, Poplar Island

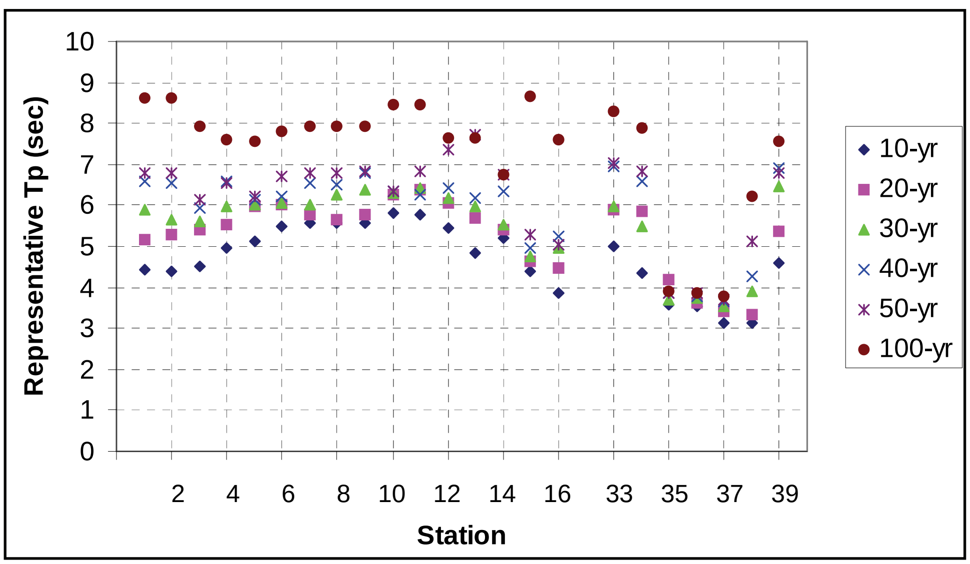


Figure 62. Return period T_p at nearshore stations, Poplar Island

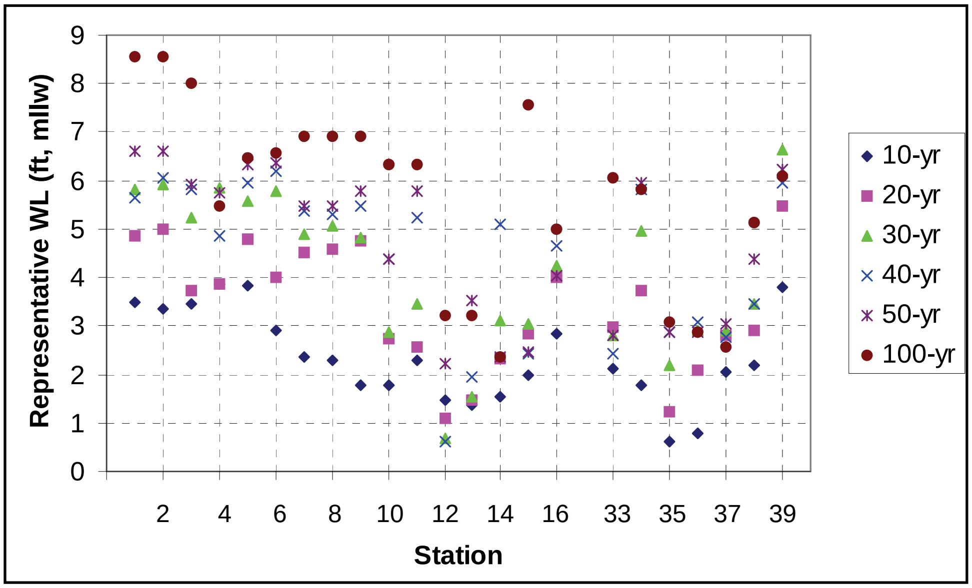


Figure 63. Return period water level at nearshore stations, Poplar Island

The last two tables in Appendix B, Tables B24 and B25, give results of an extremal analysis of water levels only for Poplar Island. The peak water levels from storms were fit to a Fisher-Tippett Type I distribution. The extremal water levels, referenced to msl, associated with northeasters from Table 16 in Chapter 3 are listed in Table B24. The extremal water levels, referenced to msl, associated with tropical storms from Table 15 in Chapter 3 are listed in Table B25. The relationship used here for Poplar Island tidal datums is msl = 0.230 m mllw. Note that the extremal water levels for shorter return periods may be less than the maximum spring tidal level because the storms do not typically occur on a maximum tide and the shorter return period storm surges are not very high. Extremal analysis of all water levels was not part of the study reported here.

Structural Analysis

The structural analysis of Poplar Island is composed of two primary parts: (a) preliminary analysis using only the historical waves and water levels, and (b) final design using the simulated waves and water levels. Both analyses use the program LC_COST_REV as the computation engine for the design sections and life-cycle response. For the preliminary design, the program was run for a large number of parametric permutations with only the historical wave and water level time series. The historical wave and water level time series was reordered so the most recent years were first in the life cycle. In this way, the most recent years carried more weight. These results narrowed the focus of the study. The final empirical life-cycle simulation (ELS) analysis using the empirically simulated waves and water levels was conducted for a narrow range of parametric permutations.

The primary alternative had the general geometry of cross sections constructed for Phases I and II as shown in Figure 58. An armored crest alternative was analyzed as shown in Figure 59. The upland cell configuration shown in Figure 60 was an alternative proposed for the northeastern cells. The potential upland sections are near sta 38. The layer thicknesses were assumed to be $2D_{a50}$ for the armor, $2D_{u50}$ for the filter layer, 0.3 m (1 ft) for the bedding layer, 20 cm (8 inches) for the rock roadway, and $2D_{ta50}$ for the toe armor. Here $D_{a50} = (V_{50})^{1/3} = (W_{50}/\gamma_r)^{1/3}$ is the nominal diameter of the armor corresponding to the 50 percent exceedance level on the weight distribution curve. Similarly, D_{u50} is the filter layer 50 percent exceedance nominal diameter, and D_{ta50} is the toe armor 50 percent exceedance nominal diameter. The armored crest detail includes a single layer of armor across the crest with a full $2D_{u50}$ thick filter layer and a 0.3-m (1-ft) thick bedding layer. The three layers are tied back into the fill material at the lee side of the roadway extending down to mllw.

The filter layer thickness under the toe armor was determined by fixing the toe crest elevation at +0.3 m (1 ft) mllw and requiring the crest armor thickness to be $2D_{ta50}$. The filter layer was sized such that $W_u = W_a/10$. The bedding material was assumed to be crushed gravel-sized material.

The analysis in LC_COST_REV contains two primary failure modes: armor stability and overtopping erosion of the crest. Toe damage is computed but the toe stability equation is not reliable because it often does not converge. Therefore, toe damage is not considered as a failure mode. At this time, there is no reliable toe damage progression model.

Overtopping rate limits of 0.05 cu m/sec/m (4.0 gal/sec/ft), corresponding to extensive damage on an unarmored crest; 0.2 cu m/sec/m (16.1 gal/sec/ft), corresponding to damage on a paved crest; and 1,000 cu m/sec/m (80,519 gal/sec/ft), corresponding to a fictitious damage limit on a heavily armored crest, were used in the optimization. The *Coastal Engineering Manual* and the *CIRIA Rock Manual* (1991) both contain equations for crest armor stability. However, the empirical equations provide nothing better than crude estimates of stable stone weight. All applicable equations and figures were investigated to determine stable crest armor requirements for Poplar Island. Although estimates of stable weight varied by a factor of four or more between the different methods, three of the methods agreed to within roughly 20 percent and the average estimated stone weights were within roughly 10 percent of the primary structure armor weight. Therefore, in this study, we have assumed that the *heavily armored* crest is armored with a single layer of main armor. The crest armoring options are summarized as follows:

a. **Unarmored**. Gravel on geotextile, overtopping limit = 0.05 cu m/sec/m (4.0 gal/sec/ft). The consequence of exceeding the overtopping limit is structure breach.

b. **Paved**. Asphalt pavement, overtopping limit = 0.20 cu m/sec/m (16.1 gal/sec/ft). The consequence of exceeding the overtopping limit is structure breach.

c. **Heavily armored**. Single layer of main armor on filter layers, overtopping limit = 1,000 cu m/sec/m (80,519 gal/sec/ft). The overtopping limit will never be reached. The stone is sized for 2 percent displacement by count for return-period wave conditions.

Preliminary analysis using historical waves and water levels

The preliminary structural optimization for Poplar Island for the historical wave climate is separated into two parts: optimization for least cost and optimization for fewest repairs.

The figures and tables in Appendix B show the significant wave height, peak period, and depth as a function of return period from the extremal wave height analysis discussed previously. Each design analysis station is shown on a separate plot. As discussed in Chapter 5, the wave heights were computed from an extremal analysis, and the wave periods and water depths were bin-averaged for each wave height. In general, the wave heights are lower than reported in previous Poplar Island design reports. In Appendix C, Figures C1-C8 show the stable main armor weight as a function of return period for each design analysis station of the northern expansion of Poplar Island computed using Equations 30-34 and the extremal waves from Appendix B. In general, the results for a seaward structure slope of cot $\alpha = 3.0$ are shown. The results for cot $\alpha = 2.5$ for sta 33 are shown for comparison. The shallow slopes were based on the requirement of a stable slope during construction.

The figures show results for the stability relations listed in Chapter 6 (Melby and Hughes 2004; van der Meer 1987; Hudson 1959). The Melby and Hughes and van der Meer relations agree very well, as they are based on the same data set. The relative magnitude of the methods varies depending on a number of factors. The Hudson equation is not conservative for this analysis because it consistently underpredicts the armor stone size. The Hudson and van der Meer equations do not include the effect of water depth. Therefore, they are not optimal for shallow water applications like Poplar Island. The Melby and Hughes relation is presumed to be more accurate than the other two equations because wave nonlinearity in shallow water is included explicitly. Therefore, here, the Melby and Hughes stability relations are used for main armor stability unless otherwise stated (Equations 30-34).

The stone sizes in Appendix C based on the extremal waves were used to develop potential design cross sections. The armor, underlayer, and toe stone weights are summarized in Table 30 for a structure slope of cot $\alpha = 3.0$.

In the optimization, these sections were exposed to life cycles of either historical waves and water levels (preliminary analysis) or simulated and historical waves and water levels (final analysis) in order to calculate the life-cycle response of the structure. The fixed input parameters for the analysis were summarized in Table 28, and parameters that were varied were listed in Table 29 in Chapter 6. The mllw depths (Table 24) and extremal wave parameters (Appendix B) were constant for all simulations but unique for each station.

Preliminary analysis based on minimum cost using historical waves and water levels

For the preliminary analysis phase, LC_COST_REV was run for a large number of parametric permutations in order to reduce the scope of the investigation. Only the historical waves and water levels were used for this part.

The potential design cross sections were determined for the extremal waves as described above.

The fixed and variable input parameters for this analysis were summarized in Tables 28 and 29. The depths relative to mllw and the extremal wave parameters were unique for each station and were input for each run.

Table 30
Stable Stone Weights for all Stations and Several Return Periods for Poplar Island

Station	Return Period					
	10 year	**20 year**	**30 year**	**40 year**	**50 year**	**100 year**
Main Armor Stable Stone Weight W_{a50} in N (lb)						
33	3,194 (718)	5,307 (1,193)	6,325 (1,422)	8,016 (1,802)	8,985 (2020)	21,271 (4,782)
34	2,767 (622)	5,240 (1,178)	6,170 (1,387)	10,004 (2,249)	12,673 (2849)	24,732 (5,560)
35	1,415 (318)	2,304 (518)	2,491 (560)	2,905 (653)	3,172 (713)	3,332 (749)
36	1,392 (313)	2,015 (453)	2,522 (567)	2,856 (642)	3,221 (724)	3,510 (789)
37	503 (113)	801 (180)	1,019 (229)	1,188 (267)	1,277 (287)	1,681 (378)
38	338 (76)	534 (120)	730 (164)	899 (202)	1,174 (264)	1,810 (407)
39	316 (71)	730 (164)	1,441 (324)	2,104 (473)	2,571 (578)	5,053 (1,136)
Underlayer Stable Stone Weight W_{u50} in N (lb)						
33	320 (72)	529 (119)	632 (142)	801 (180)	899 (202)	2,126 (478)
34	276 (62)	525 (118)	618 (139)	1,001 (225)	1,268 (285)	2,473 (556)
35	142 (32)	231 (52)	249 (56)	289 (65)	316 (71)	334 (75)
36	138 (31)	200 (45)	254 (57)	285 (64)	320 (72)	351 (79)
37	49 (11)	80 (18)	102 (23)	120 (27)	129 (29)	169 (38)
38	36 (8)	53 (12)	71 (16)	89 (20)	116 (26)	182 (41)
39	31 (7)	71 (16)	142 (32)	209 (47)	258 (58)	507 (114)
Toe Armor Stable Stone Weight W_{t50} in N (lb)						
33	1,632 (367)	2,718 (611)	3,243 (729)	4,101 (922)	4,608 (1,036)	10,894 (2,449)
34	1,415 (318)	2,687 (604)	3,154 (709)	5,120 (1151)	6,486 (1,458)	12,664 (2,847)
35	1,352 (304)	1,174 (264)	1,277 (287)	1,495 (336)	1,619 (364)	1,699 (382)
36	996 (224)	1,028 (231)	1,294 (291)	1,463 (329)	1,650 (371)	1,797 (404)
37	258 (58)	405 (91)	525 (118)	552 (124)	623 (140)	859 (193)
38	169 (38)	276 (62)	374 (84)	454 (102)	605 (136)	930 (209)
39	160 (36)	374 (84)	743 (167)	1,076 (242)	1,317 (296)	2,589 (582)

Present worth as a function of return period was calculated based on a value of inflation of 0.0. The least-quantity cross section was always the least cost. This is because there is no cost penalty for breaching in latter years with no inflation. Table 31 summarizes results of the cost minimization analysis for each station. Several conclusions can be drawn from these results as follows:

a. For several stations, the costs are independent of the stone size past the minimum point of the cost curve (e.g., sta 35, 36, 37, and 38).

b. The crest armoring always was more costly than an unarmored crest.

c. The steepest structure slope of cot α = 2.5 was always the least cost.

d. The minimum-cost return period steadily increases as one proceeds clockwise around the island from sta 33 to 38, where it decreases again.

e. The stable armor stone size decreases as one proceeds clockwise around the island from sta 33.

f. The largest armor stone size is at sta 33 and is 956 lb.

g. The stable armor stone size decreases dramatically at sta 37 to 333 lb.

h. It appears that the design can be reduced to two unique cross sections: sta 33-36 using the sta 33 cross section, and sta 37-38 combined using the sta 37 cross section. Station 39 may also need to be separated because of the southern exposure and larger stone size and crest height required over sta 37.

i. Costs are nearly constant over the range of fixed costs used.

j. There is not a significant cost penalty for shallower structure slopes because the stone size decreases as the structure slope decreases and the number of failures resulting from overtopping also decrease.

k. There is not a significant cost penalty for higher crests because of the decrease in repair costs.

l. Including inflation and including some extraneous costs associated with failure, like cleanup, will change the relative costs. For more significant repair costs, the more reliable structures with larger stones, armored crests, and higher crests will likely be more economical.

Preliminary analysis based on minimum repairs and historical waves and water levels

A number of trials were run with program LC_COST_REV in order to define damage throughout the life of the structure. This analysis facilitates selecting the option with the most reliability. Inflation of i = 0.03 was included in this analysis. The input wave conditions were only the historical. The results are summarized in the following two sections.

Table 31
Preliminary Least-Cost Analysis Results for Historical Waves and Water Levels

Station	Overtopping Limit, cu m/sec/m (gal/sec/ft)	Crest Height, m (ft)	Slope cot α	Return Period, years	Total Cost
33	0.05 (4)	2.74 (9)	2.5	10	$4,200/m ($1,280/ft)
	0.2 (16.1)	2.44 (8)	2.5	10	$4,535/m ($1,382/ft)
34	0.05 (4)	3.35 (11)	2.5	10	$4,469/m ($1,362/ft)
	0.2 (16.1)	2.59 (8.5)	2.5	10	$4,729/m ($1,441/ft)
35	0.05 (4)	2.44 (8)	2.5	25	$3,827/m ($1,166/ft)
	0.2 (16.1)	2.44 (8)	2.5	25	$4,232/m ($1,290/ft)
36	0.05 (4)	2.44 (8)	2.5	30	$3,578/m ($1,091/ft)
	0.2 (16.1)	2.44 (8)	2.5	35	$4,009/m ($1,222/ft)
37	0.05 (4)	2.44 (8)	2.5	30	$3,280/m ($1,000/ft)
	0.2 (16.1)	2.44 (8)	2.5	30	$3,588/m ($1,094/ft)
38	0.05 (4)	2.44 (8)	2.5	20	$2,716/m ($828/ft)
	0.2 (16.1)	2.44 (8)	2.5	20	$2,917/m ($889/ft)
39	0.05 (4)	2.59 (8.5)	2.5	10	$2,144/m ($653/ft)
	0.2 (16.1)	2.44 (8)	2.5	10	$2,322/m ($708/ft)

Stability damage analysis

To isolate failure of the structure from armor instability, the crest elevation was set just high enough so that there would be no failures resulting from overtopping. The constant parameters for this analysis are listed in Table 32. Table 33 and Figure 64 summarize the breaches resulting from instability. This preliminary analysis indicates that there is little possibility for failure due to armor instability if the structure cross sections are designed for return periods of 35 years or greater. The following section provides a brief summary of damage for each station.

Station 33. A return period of 5 years has seven breaches resulting from stability. For a 10-year return period, the number of breaches drops to three. For a return period of 15 years, the number of breaches drops to one, and there are no breaches for longer return periods.

Station 34. A return period of 5 years has five breaches resulting from stability. For 10- and 15-year return periods, the number of breaches drops to three. For return periods of 20-30 years, the number of breaches drops to two; for 35 years, there is one breach; and there are no breaches for return periods of 40 years and longer.

Station 35. A return period of 5 years has six breaches resulting from stability. There are no breaches for return periods longer than 5 years.

Table 32
Fixed Parameter Values for Armor Damage Analysis

Parameter	Variable	Value
Permeability	P	0.1
Porosity	Por	0.38
Stone specific gravity	S_r	2.578
Stone density	ρ_r	2.644 tonne/cu m (165 pcf)
Minor repair limit	S_M	8
Breach repair limit	S_B	18
Minor repair time limit	-	180 days
Breach repair time limit	–	120 days
Roughness parameter	γ_b	0.55
Crest width	B	7.62 m (25 ft)
Upper structure slope	α_2	1V:3H
Toe berm height	d_B	+0.305 m (1 ft) mllw
Toe berm seaward slope	cot φ	2
Toe berm leeward slope	cot β	1.5
Toe berm crest width	-	$4D_{toe}$
Toe armor thickness	-	$2D_{toe}$
Allowable main armor damage	S	1.0
Allowable toe damage	N_{od}	1.0
Number of waves for zero damage	N_z	7,000
Inflation or escalation rate	i	0.03
Interest rate	R	0.05375
Economic life	N	50 years
Armor material unit cost	-	$56/tonne ($50.4/ton)
Filter material unit cost	-	$39/tonne ($35.1/ton)
Bedding material unit cost	-	$44/tonne ($39.6/ton)
Quarry-run material unit cost	-	$44/tonne ($39.6/ton)
Geotechnical material unit cost	-	$4.78/sq m ($0.44/sq ft)
Lag before initial construction	Lag	2 years
Fixed first cost	FFC/L_s	$500/m ($152/ft)
Fixed repair cost	RFC/L_r	$2,500/m ($762/ft)
Structure slopes	Cot α	3.0
Ratio of repair length to section length	L_r/L_s	0.3

Table 33
Number of Breaches Due to Armor Instability as Function of Return Period for Historical Wave Conditions for Poplar Island

Station	Return Period										
	5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**100**
33	7	3	1	0	0	0	0	0	0	0	0
34	5	3	3	2	2	2	1	0	0	0	0
35	6	0	0	0	0	0	0	0	0	0	0
36	8	1	0	0	0	0	0	0	0	0	0
37	6	0	0	0	0	0	0	0	0	0	0
38	5	1	1	0	0	0	0	0	0	0	0
39	7	4	3	2	2	0	0	0	0	0	0

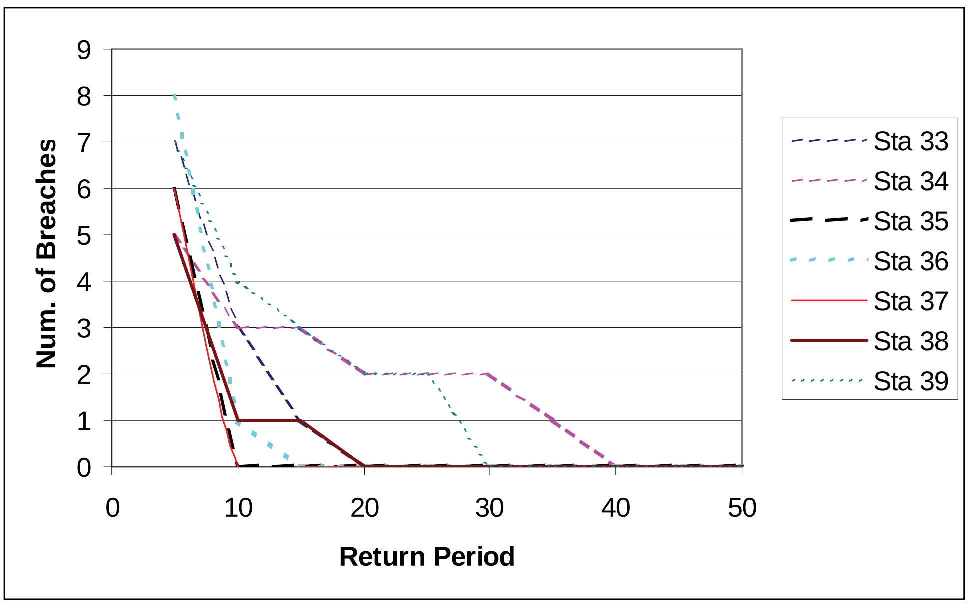


Figure 64. Number of breaches due to armor instability as function of return period for historical wave conditions for Poplar Island

Station 36. A return period of 5 years has eight breaches resulting from stability. For a return period of 10 years, the number of breaches drops to one, and there are no breaches for longer return periods.

Station 37. A return period of 5 years has six breaches resulting from stability. There are not breaches for return periods longer than 5 years.

Station 38. A return period of 5 years has five breaches resulting from stability. For return periods of 10 and 15 years, the number of breaches drops to one, and there are no breaches for longer return periods.

Station 39. A return period of 5 years has seven breaches resulting from stability. For a 10-year return period, the number of breaches drops to four. For a return period of 15 years, the number of breaches drops to three. For 20-25 years, the number of breaches drops to two, and there are no breaches for return periods of 30 years and longer.

Overtopping failure

To isolate breach failures due to overtopping as a function of crest height, only return periods greater than 30 years were considered. The number of breaches due to overtopping as a function of return period was constant for these longer return periods. The constant parameters for this analysis are listed in Table 32.

For all stations, overtopping was analyzed for heavily armored, paved, and unarmored crests. Here we assume that for a heavily armored crest, if crest armoring equivalent in size to the main structure armor is used, then there will be no damage due to overtopping. Breaches from overtopping are summarized in the following lists and in Tables 34 and 35.

Station 33.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For an 8- to 11-ft crest height, two breaches occur for Hurricanes Isabel and Hazel. For crest heights of 11.5 ft or higher, no breaches occur.

c. Unarmored crest. For an 8-ft crest height, five breaches occur. For a 9-ft crest height, three breaches occur. For a 10- to 13-ft crest height, two breaches occur, for Hurricanes Isabel and Hazel. For crest heights of 14 ft or higher, no breaches occur.

d. Hurricane Isabel was in 2003 and Hurricane Hazel was in 1953. Other breaches occurred from two other extratropical storms in the mid-1950s and early 1960s.

Station 34.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For an 8-ft crest height, three breaches occur. For a 9- to 11.5-ft crest height, two breaches occur for Hurricanes Isabel and Hazel. For a 12-ft crest height, one breach occurs. For crest heights of 13 ft or higher, no breaches occur.

c. Unarmored crest. For an 8- to 9-ft crest height, five breaches occur. For a 10- to 14-ft crest height, two breaches occur, for Hurricanes Isabel and Hazel. For crest heights of 15 ft or higher, no breaches occur.

Table 34
Number of Breaches of Unarmored Crest Due to Overtopping for Return Period of 35 Years as Function of Crest Height for Historical Wave Conditions for Poplar Island

Station	Crest Height, ft								
	8	9	10	11	12	13	14	15	16
33	5	3	2	2	2	2	0	0	0
34	5	5	2	2	2	2	2	0	0
35	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0
37	0	0	0	0	0	0	0	0	0
38	2	0	0	0	0	0	0	0	0
39	2	2	0	0	0	0	0	0	0

Table 35
Number of Breaches of Paved Crest due to Overtopping for Return Period of 35 Years as Function of Crest Height for Historical Wave Conditions for Poplar Island

Station	Crest Height, ft								
	8	9	10	11	12	13	14	15	16
33	2	2	2	2	0	0	0	0	0
34	3	2	2	2	1	0	0	0	0
35	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0
37	0	0	0	0	0	0	0	0	0
38	0	0	0	0	0	0	0	0	0
39	2	2	0	0	0	0	0	0	0

Station 35.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For crest heights of 8 ft or higher, no breaches occur.

c. Unarmored crest. For crest heights of 8 ft or higher, no breaches occur.

Station 36.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For crest heights of 8 ft or higher, no breaches occur.

c. Unarmored crest. For crest heights of 8 ft or higher, no breaches occur.

Station 37.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For crest heights of 8 ft or higher, no breaches occur.

c. Unarmored crest. For crest heights of 8 ft or higher, no breaches occur.

Station 38.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For crest heights of 8 ft or higher, no breaches occur.

c. Unarmored crest. For an 8-ft crest height, two breaches occur. For crest heights of 9 ft or higher, no breaches occur.

Station 39.

a. Heavily armored crest. No breaches resulting from overtopping.

b. Paved crest. For an 8- to 9-ft crest height, two breaches occur. For a crest heights of 10 ft or higher, no breaches occur.

c. Unarmored crest. For an 8-ft crest height, three breaches occur. For a 9- to 10.5-ft crest height, two breaches occur. For an 11-ft crest height, one breach occurs. For crest heights of 11.5 ft or higher, no breaches occur.

Summary of structural analysis based on historical waves and water levels

The results above do not provide a satisfactory design because the crest height required to prevent damage is overly high. The toe stone size is also quite large. In addition, armoring the crest with primary armor was considered to be unacceptable at this stage. Based on these results and discussions with Baltimore District personnel, preliminary recommendations for design were developed with the caveat that they would be adjusted during the final analysis phase.

Stations 33-36, eastern reach of Phase III expansion.

a. Return period for cross-sectional design: 45 years.

b. Stone sizes based on sta 34: W_{a50} = 1.12 tonnes (2,500 lb).

c. Crest height = 3.20 m (10.5 ft).

d. Structure slope = 1V:3.0H.

Stations 37-39, northern and eastern reaches of Phase III expansion.

a. Return period for cross-sectional design: 35 years.

b. Stone sizes based on sta 39: W_{a50} = 0.158 tonnes (350 lb).

c. Crest height = 3.20 m (10.5 ft).

d. Structure slope = 1V:3.0H.

Final Analysis Using ELS Simulations

The previous sections provided focus for a final analysis using the ELS technique. For this analysis, 50 simulations of a 50-year wave and water level climate were generated for each design analysis station. Only four of the stations will be described in this summary. Each wave time series was run through LC_COST_REV for three crest heights, all relative to mllw: sta 33, 34, and 39: 2.90 m (9.5 ft), 3.20 m (10.5 ft), and 3.505 m (11.5 ft); sta 37: 2.28 m (7.5 ft), 2.90 (9.5 ft), and 3.20 m (10.5 ft). These stations were considered representative of broad reaches around the northern expansion of Poplar Island. The fixed parameters for this portion of the study are listed in Table 36.

All empirical simulations of wave and water level life cycles were compared to the historical values to assure that they were statistically similar. In particular, time series, histograms, and cumulative distributions were plotted for H_s, T_p, water level, and wave direction for each simulation. In addition, the upper tails of the histograms and of the empirical cumulative distributions were analyzed to assure that the extreme values were being reproduced. In all cases, the distributions of historical time series were very well reproduced in the simulations. Figures 65-67 show distributions of historical and simulated H_s, T_p, and water level, respectively, for sta 33. As can be seen, there is little difference between the historical and simulated distributions. Only T_p differs slightly at the upper and lower tails. This small difference produced no noticeable effect on the damage predictions. For all stations and all simulations, the empirical distribution fits shown in Figures 65-67 were typical.

Table 36
Fixed Parameter Values for Final ELS Analysis

Parameter	Variable	Value
Permeability	P	0.1
Porosity	Por	0.38
Stone specific gravity	S_r	2.578
Stone density	ρ_r	2.644 tonne/cu m (165 pcf)
Minor repair limit	S_M	8
Breach repair limit	S_B	18
Minor repair time limit	-	180 days
Breach repair time limit	–	120 days
Roughness parameter	γ_b	0.55
Crest width	B	7.62 m (25 ft)
Lower structure slope	α	1V:3H
Upper structure slope	α_2	1V:3H
Toe berm height	d_B	+0.305 m (1 ft) mllw
Toe berm seaward slope	cot φ	2
Toe berm leeward slope	cot β	1.5
Toe berm crest width	-	$4D_{toe}$
Toe armor thickness	-	$2D_{toe}$
Overtopping limit		0.05 cu m/sec/m
Allowable main armor damage	S	1.0
Allowable toe damage	N_{od}	1.0
Number of waves for zero damage	N_z	7000
Inflation or escalation rate	i	0.03
Interest rate	R	0.05375
Economic life	N	50 years
Armor material unit cost	-	$56/tonne ($50.4/ton)
Filter material unit cost	-	$39/tonne ($35.1/ton)
Bedding material unit cost	-	$44/tonne ($39.6/ton)
Quarry-run material unit cost	-	$44/tonne ($39.6/ton)
Geotechnical material unit cost	-	$4.78/sq m ($0.44/sq ft)
Lag before initial construction	Lag	0 years
Fixed first cost	FFC/L_s	$500/m ($152/ft)
Fixed repair cost	RFC/L_r	$2,500/m ($762/ft)
Ratio of repair length to section	L_r/L_s	0.3

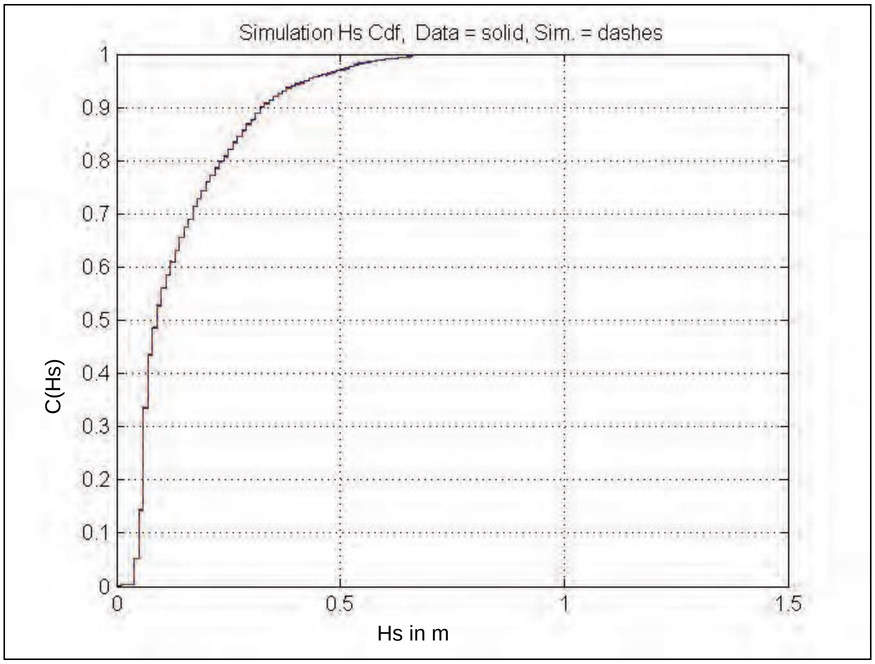


Figure 65. Cumulative distribution function for historical H_s and one 50-year simulation for sta 33

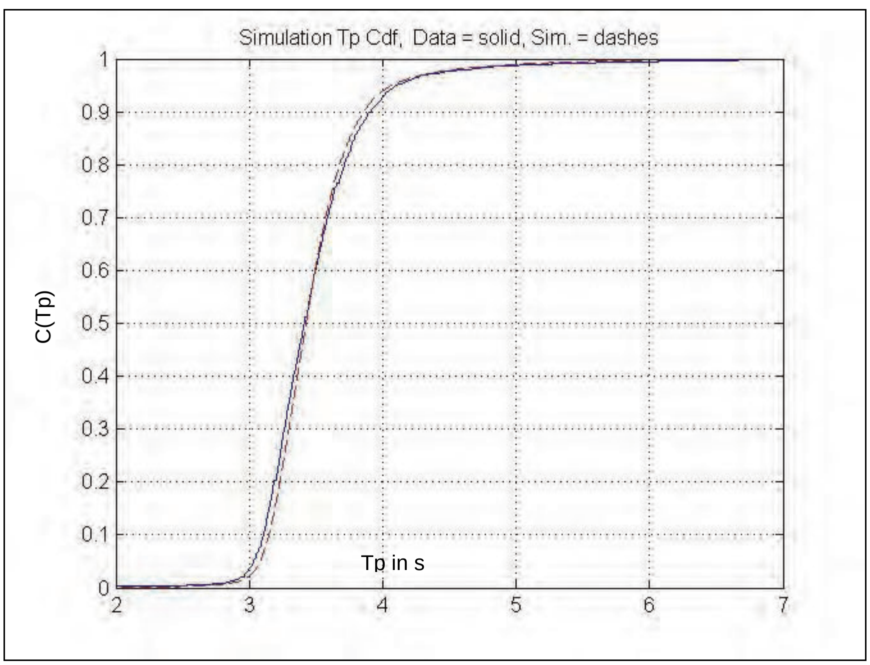


Figure 66. Cumulative distribution function for historical T_p and one 50-year simulation for sta 33

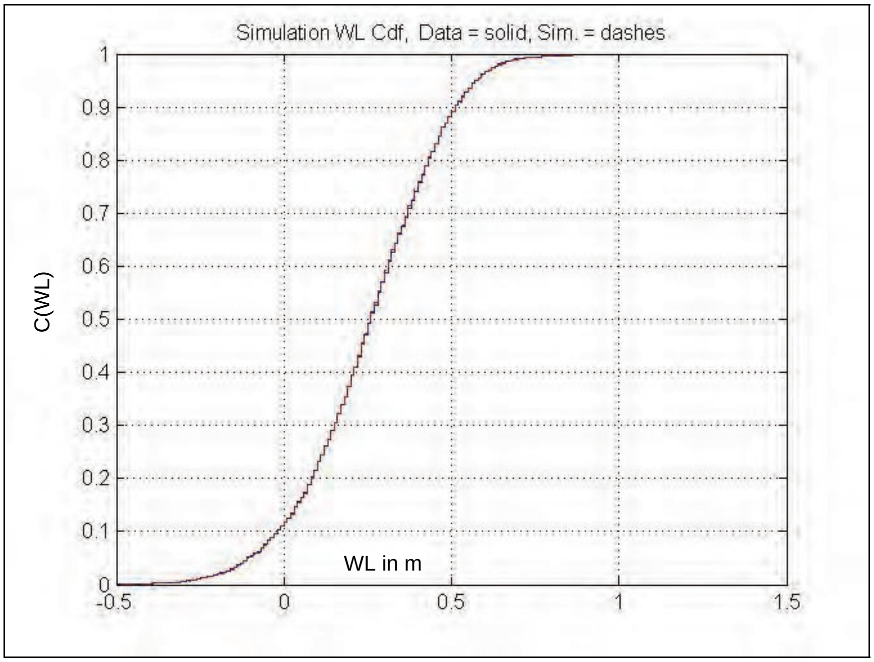


Figure 67. Cumulative distribution function for historical water level, relative to mllw, and one 50-year simulation for sta 33

The ELS simulations produced a significant amount of data. These data were reduced using several custom FORTRAN and Matlab programs. The primary focus of the post-simulation analysis was to determine the probability of damage to the structure cross sections at representative sta 33, 34, 37, and 39. Empirical exceedance distributions were generated for normalized eroded cross-sectional area *S* with no repairs (*No-Repair S*), the *Number of Repairs*, and the *Present Worth Repair Cross-Sectional Cost*. The *No-Repair S* was computed in order to provide an impression of the level of damage if no repairs were done over the economic life of the structure. Note that the level of damage with no repairs can easily get to the point of being unrealistic. If $S = 20$ is exceeded, it is expected that the structure would be in the breach condition, so this parameter should be considered qualitative over values of about 20. The exceedance analysis was done for durations of 2, 5, 10, 20, 30, 40, and 50 years into the 50-year simulation.

Representative Figures 68-73 plot exceedance probability for *No-Repair S*, *Number of Repairs*, and *Present Worth Repair Cross-Sectional Cost* for a crest height of 2.90 m at sta 33. Figures 68 and 69 show exceedance distributions of *No-Repair S* for 30- and 50-year return periods, respectively. Figures 70 and 71 show exceedance distributions of *Number of Repairs* for 30- and 50-year return periods, respectively. Figures 72 and 73 show exceedance distributions of the corresponding *Present-Worth Repair Cross-Sectional Cost*. In Figures 68-73, the legend refers to the duration into the time series. Although only representative figures are shown, the other crest heights for sta 33 looked similar. There was little difference in results for the three crest heights analyzed. The analysis for Figures 68-73 was done for unarmored crests.

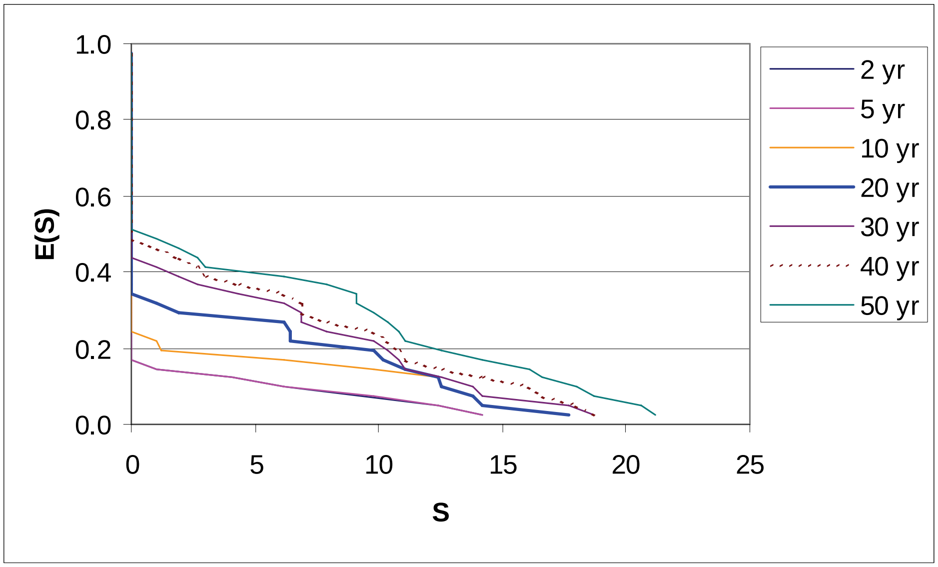


Figure 68. Exceedance probability of eroded cross-sectional area *No-Repair S* at sta 33 for crest height of 2.90 m (9.5 ft) and 30-year return-period cross section. Muliple curves show varying durations into 50-year simulation

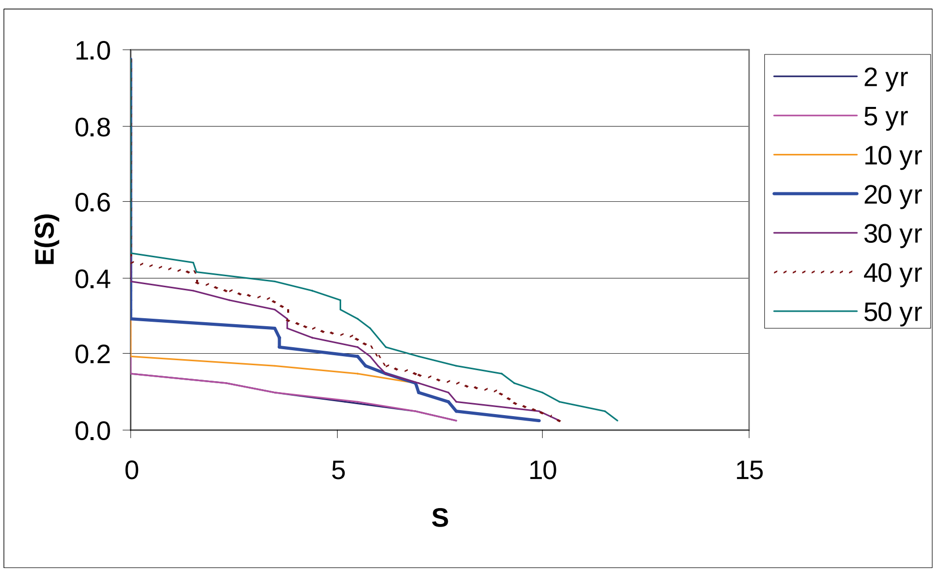


Figure 69. Exceedance probability of eroded cross-sectional area *No-Repair S* at sta 33 for crest height of 2.90 m (9.5 ft) and 50-year return-period cross section.

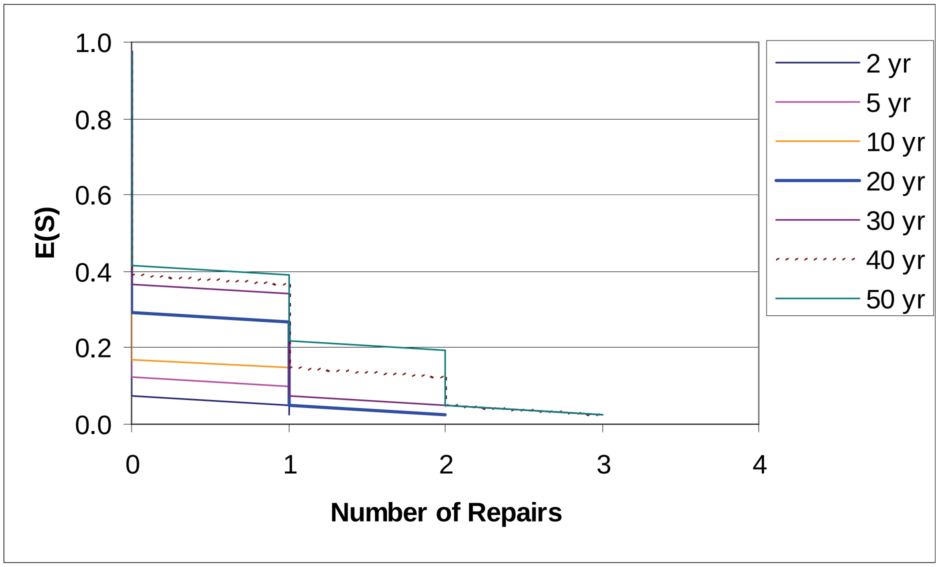


Figure 70. Exceedance probability of *Number of Repairs* at sta 33 for crest height of 2.90 m (9.5 ft) and 30-year return-period cross section

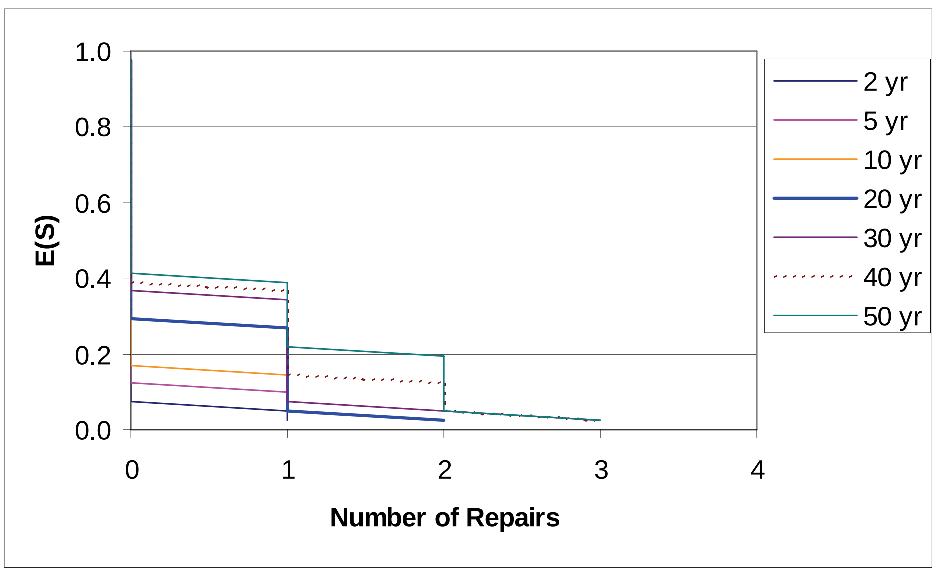


Figure 71. Exceedance probability of *Number of Repairs* at sta 33 for crest height of 2.90 m (9.5 ft) and 50-year return-period cross section

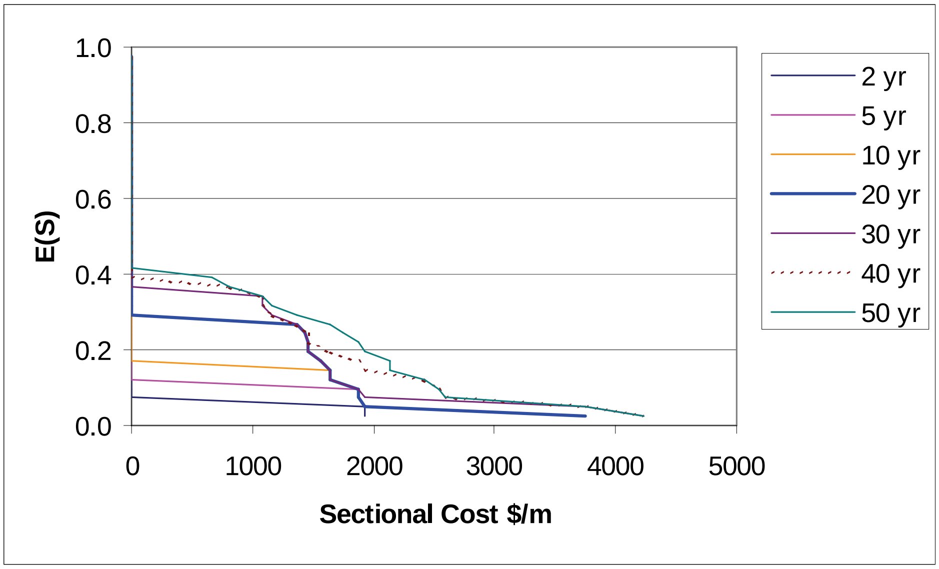


Figure 72. Exceedance probability of *Present Worth Cross-Sectional Repair Cost* per running meter at sta 33 for crest height of 2.90 m (9.5 ft) and 30-year return-period cross section

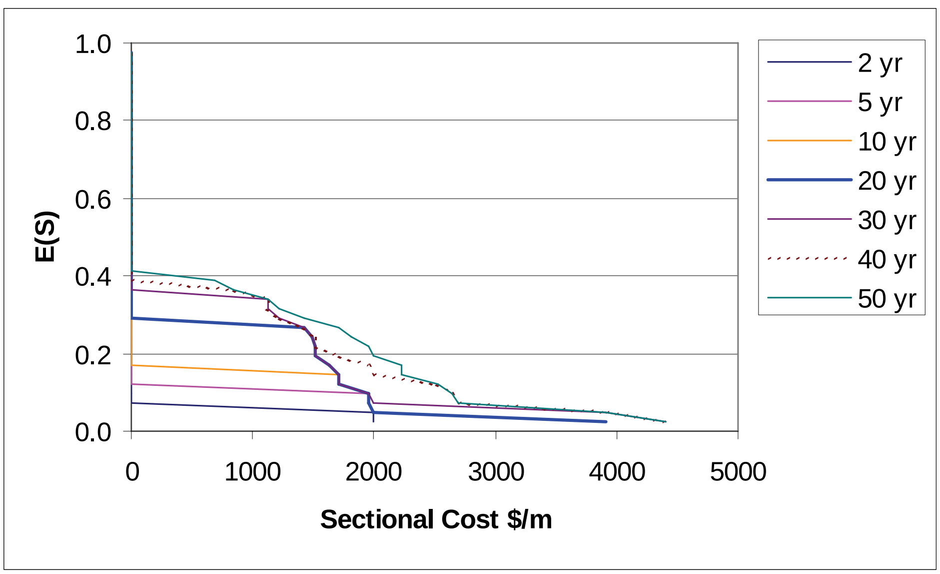


Figure 73. Exceedance probability of *Present Worth Cross-Sectional Repair Cost* per running meter at sta 33 for crest height of 2.90 m (9.5 ft) and 50-year return-period cross section

For all simulations, one would expect that the average simulation (50 percent exceedance value in Figures 68-73) would match the historical. However, the simulations tended to underpredict damage. This was because the four or so extreme hurricanes in the historical life cycle were not well reproduced in the simulations. Some simulations contained a similar number of extreme storms as in the historical case, but some did not. Very few of the simulations contained more extreme storms than the historical case. Damage is a threshold phenomenon and will only occur for extreme wave and water level conditions. So the extreme storms were more important in this study. Several strategies were attempted to improve the simulation of the extreme storms. The final accepted method separately matched the normalized distributions of wave parameters above a threshold. The final distribution tails matched reasonably well for most simulations, but most life-cycle simulations still underpredicted damage. Because of time constraints, the WELS program could not be improved for this project, so the average of the simulations that provided the best tail fit to the historical distributions was used.

The data from the exceedance analyses were further reduced to provide information for design. The following figures correspond to the average of the simulations with distributions that most closely matched the historical distributions. Figures 74-76 show the cross-sectional cost as a function of return period at sta 33 for three unarmored crest heights. The analyses shown are for the entire 50-year life cycle. Figure 74 shows the total cost for three crest heights, while Figures 75 and 76 show the cost breakdown for crest heights of 2.9 m (9.5 ft) and 3.51 m (11.5 ft), respectively. These figures show that costs generally increase with return period design and with crest height. The exception is a crest height of 3.51 m (11.5 ft). For this crest height, overtopping repairs and resulting maintenance costs were reduced.

Figure 77 shows number of repairs as a function of return period for sta 33 and several crest heights. The figure shows that there is little difference for return periods of 20 to 100 years for the range of crest heights analyzed. It is expected that environmental and cleanup costs associated with breaching would drive up the costs of repairs and result in the low-return-period sections being more expensive than shown in Figure 74. As a result, the optimal design for this station with an unarmored crest corresponds to a 40-year return period and a 3.51-m (11.5-ft) crest height.

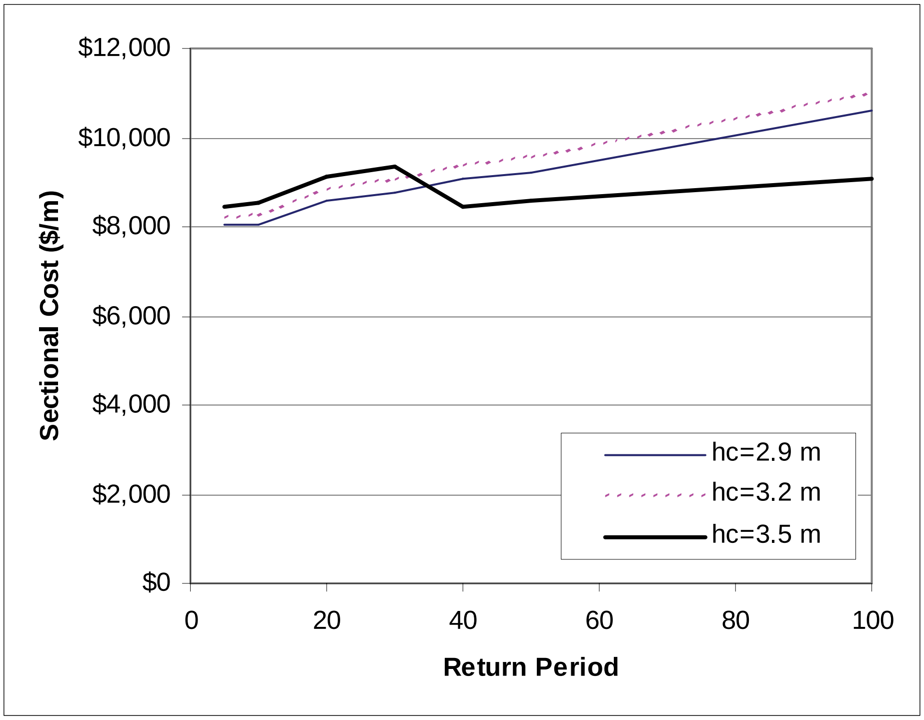


Figure 74. Total cross-sectional cost as function of return period at sta 33 for several crest heights. Crests were unarmored

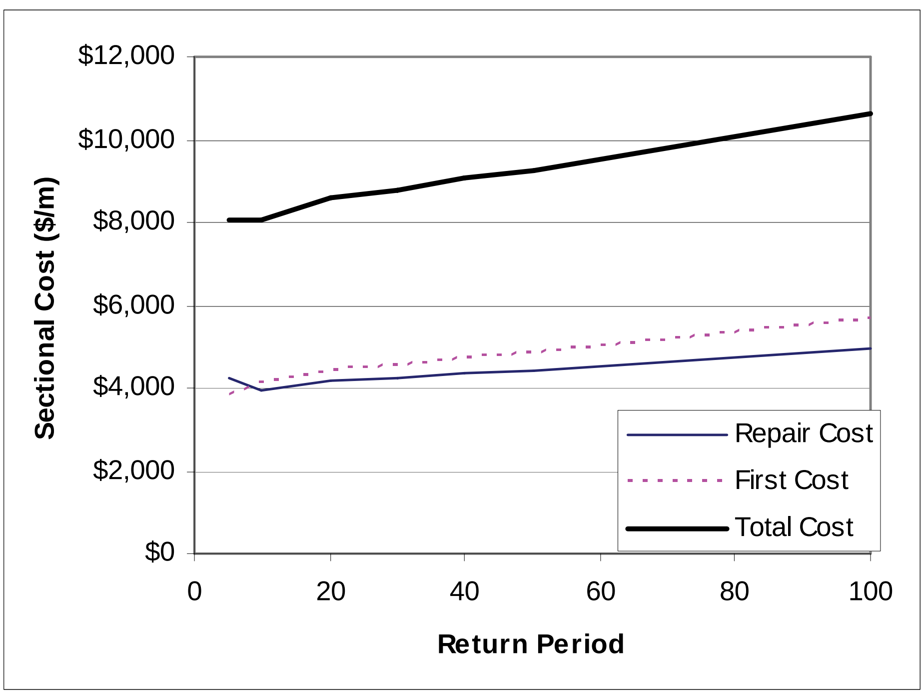


Figure 75. Cross-sectional cost as function of return period at sta 33 for unarmored crest height of 2.9 m (9.5 ft)

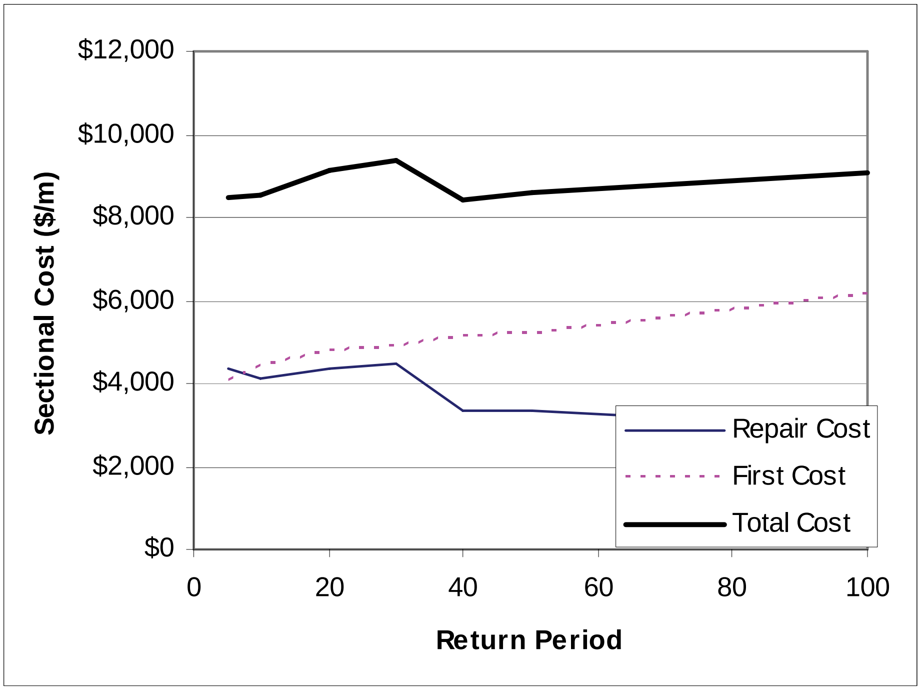


Figure 76. Cross-sectional cost as function of return period at sta 33 for unarmored crest height of 3.5 m (11.5 ft)

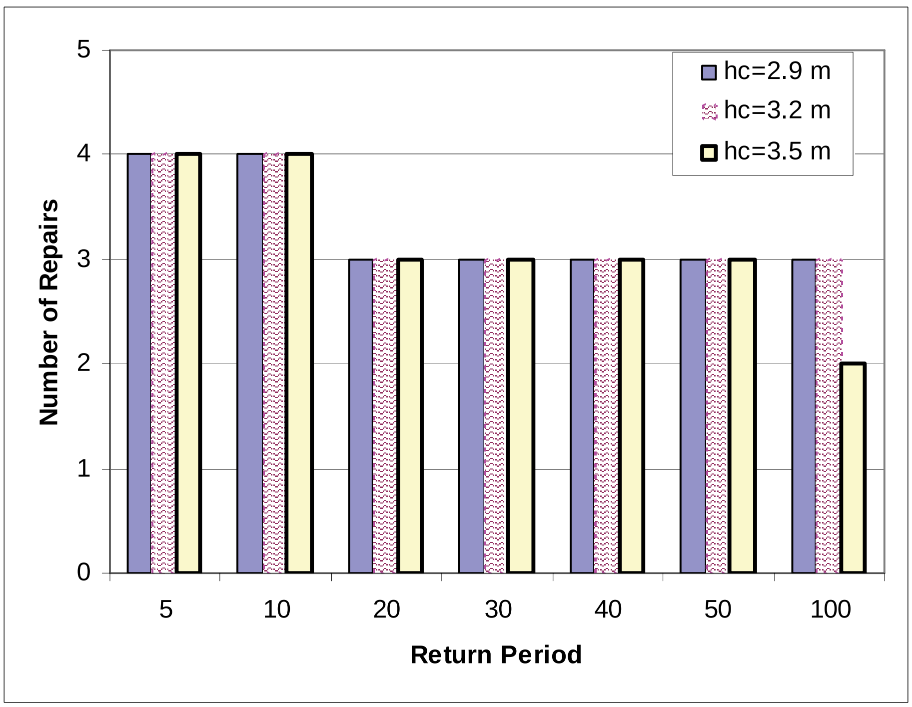


Figure 77. Total number of repairs as function of return period at sta 33 for several crest heights. Crests were unarmored

Figures 78-81 are for a heavily armored crest but are otherwise analogous to Figures 74-77. In this case, the primary structure armor has been carried over the crest as shown in Figure 59. As can be seen in Figures 78-80, the total cost has been reduced over the unarmored-crest alternative. Further, the least-cost option is now the lowest crest height of 2.90 m (9.5 ft). Finally, the total costs decrease with increasing return period, so the designs are likely to be more reliable than those with unarmored crests. These results are typical for the western side of the Phase III expansion of Poplar Island.

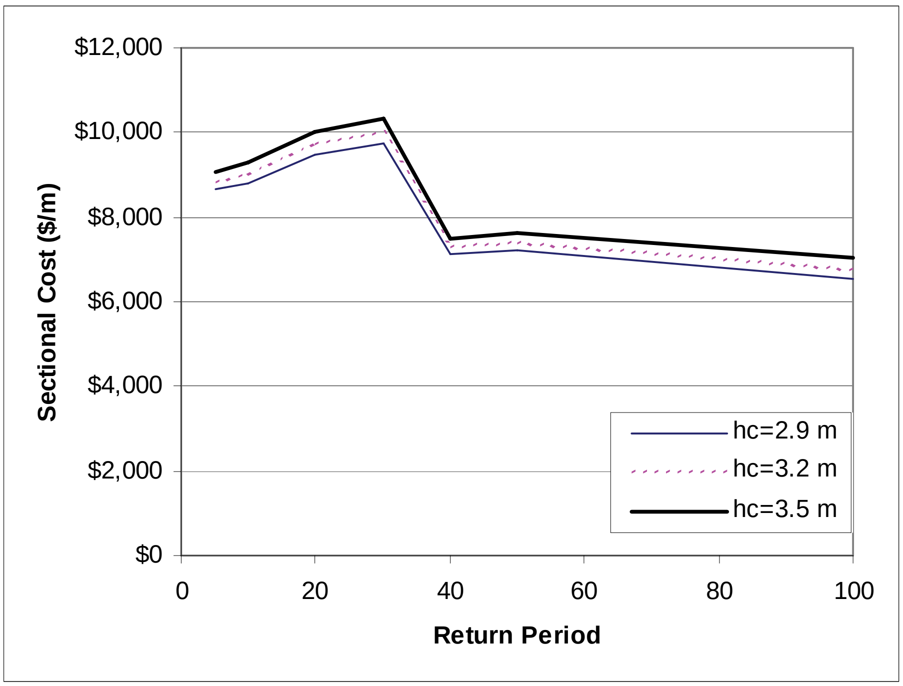


Figure 78. Total cross-sectional cost as function of return period at sta 33 for several crest heights. Crests were heavily armored

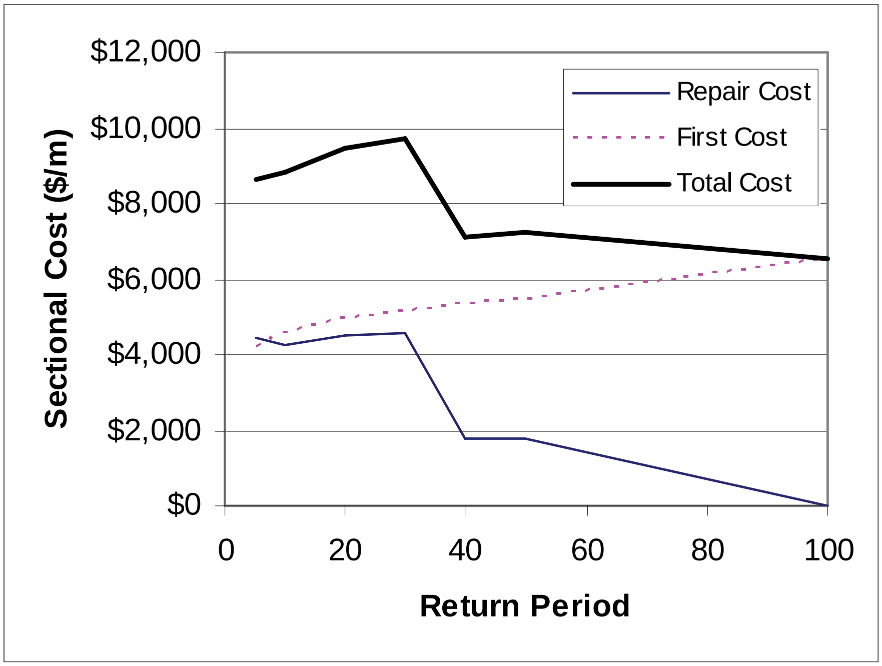


Figure 79. Cross-sectional cost as function of return period at sta 33 for heavily armored crest height of 2.9 m (9.5 ft)

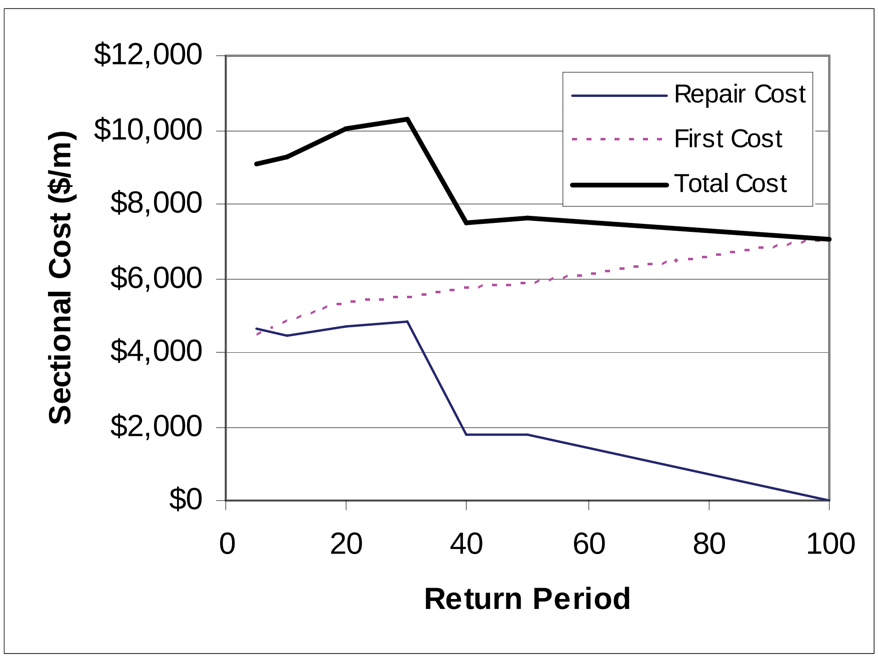


Figure 80. Cross-sectional cost as function of return period at sta 33 for heavily armored crest height of 3.5 m (11.5 ft)

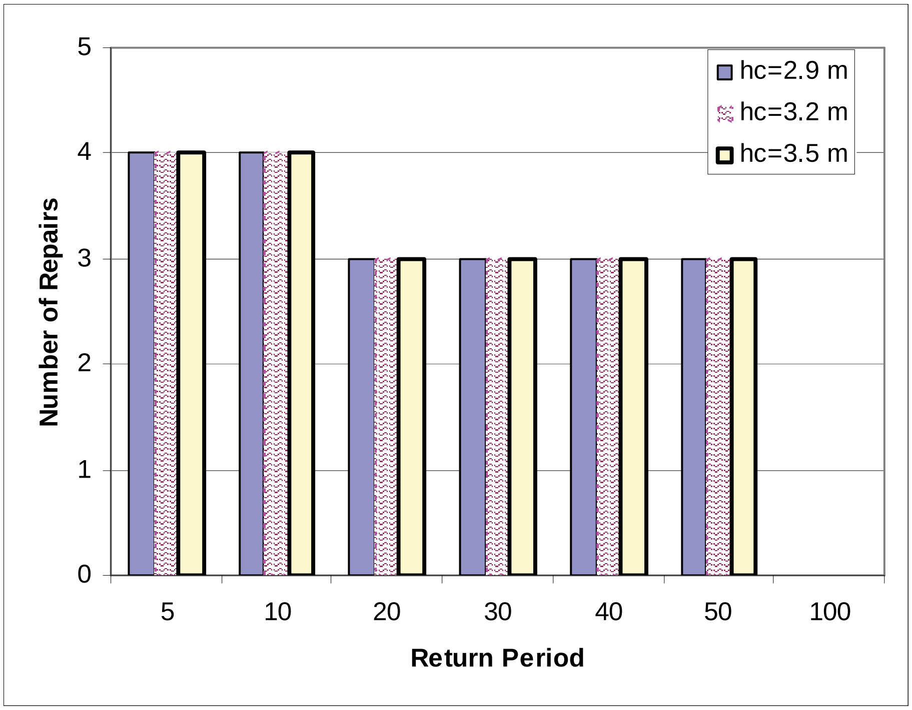


Figure 81. Total number of repairs as function of return period at sta 33 for several crest heights. Crests were heavily armored

Figure 82 shows return period versus total number of repairs from *breaches that resulted from armor instability*. Figure 83 shows return period versus total number of *minor repairs* that resulted from armor instability. Comparing these two figures yields more information about the nature of the repairs for an armored crest. As expected, with increasing stone size there are fewer breaches. For the 40-year return period there are no breaches. However, the storms that caused the breaching still produce damage. The number of minor repairs increases until the 20-year return period. Minor repairs decrease thereafter until the 100-year return period, where there are no repairs. Referring to Figure 77, for an unarmored crest, the number of overtopping repairs is mostly constant with return period. If the crest is sufficiently low, severe storms cause overtopping damage regardless of the return period design. Table 34 in the previous section showed that the unarmored crest height required to eliminate overtopping repairs was almost 4.6 m (15 ft). This crest height is unrealistic. So the unarmored crest alternative does not produce a satisfactory solution because there will always be breaches and resulting environmental damages for reasonable crest heights. However, the armored crest alternative produces a structure with no breach failures over the 50-year life cycle if the 40-year return period or greater design is used. Further, the low crest option can be used, which is both lower cost and more appealing to local property owners.

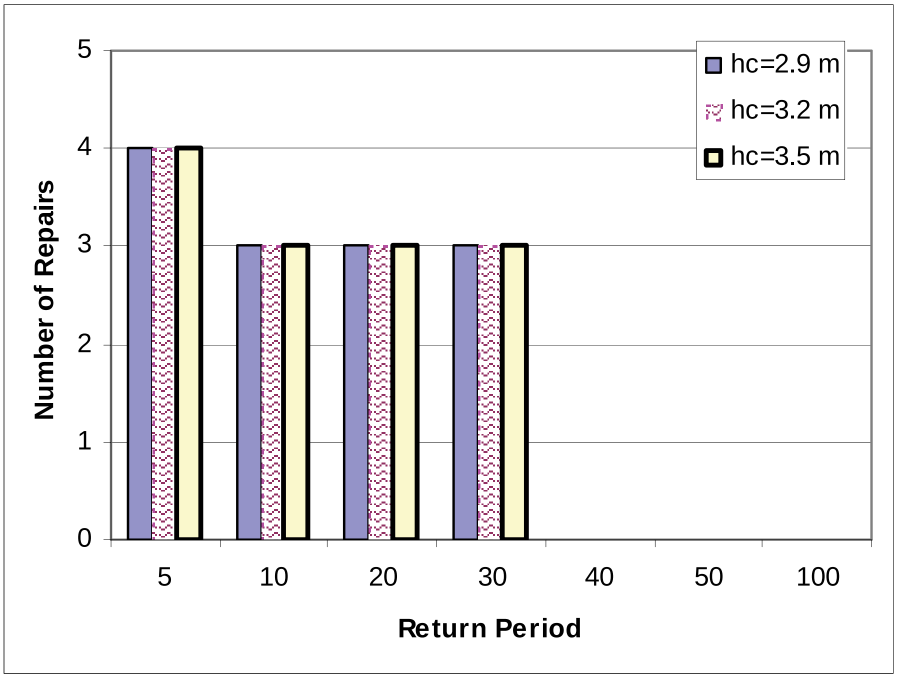


Figure 82. Total number of breach repairs due to armor instability as function of return period at sta 33 for several crest heights. Crests were heavily armored

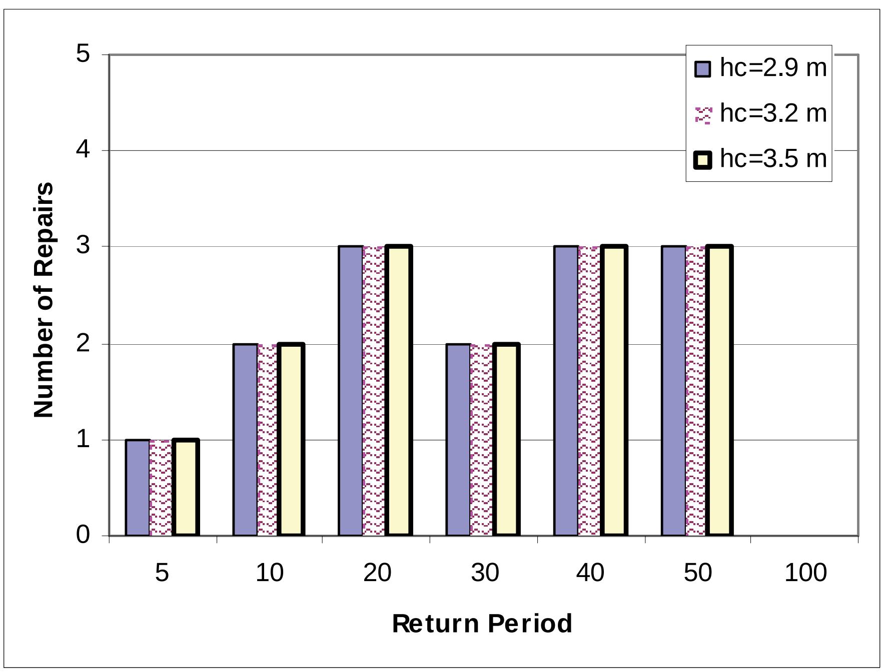


Figure 83. Total number of minor repairs due to armor instability as function of return period at sta 33 for several crest heights. Crests were heavily armored

Figures 84-87 show *Sectional Cost* and *Number of Repairs* for sta 37. The figures are similar to those shown for sta 33. The section with the unarmored crest increases in cost from the 10-year return period. The least-cost unarmored crest is the lowest at 2.29 m (7.5 ft). Figure 85 shows that all unarmored return-period design sections past 5 years require no repairs over the 50-year life. The armored crest section at sta 37, shown in Figures 86 and 87, is virtually identical in response to the unarmored crest for this station. The least-cost armored-crest section has a crest height of 2.29 m (7.5 ft) and corresponds to the 50-year return period design. Note that this crest height corresponds roughly to the highest water level predicted for the island. For sta 37, the least-cost section has an unarmored crest but the armored crest cost is only slightly greater.

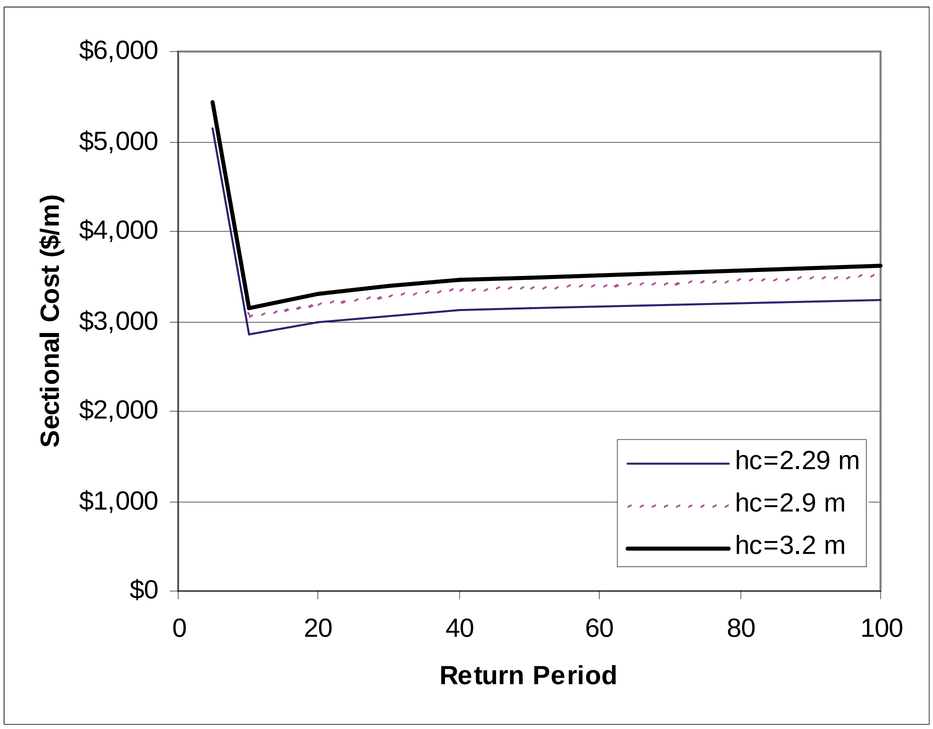


Figure 84. Total cross-sectional cost as function of return period at sta 37 for several crest heights. Crests were unarmored

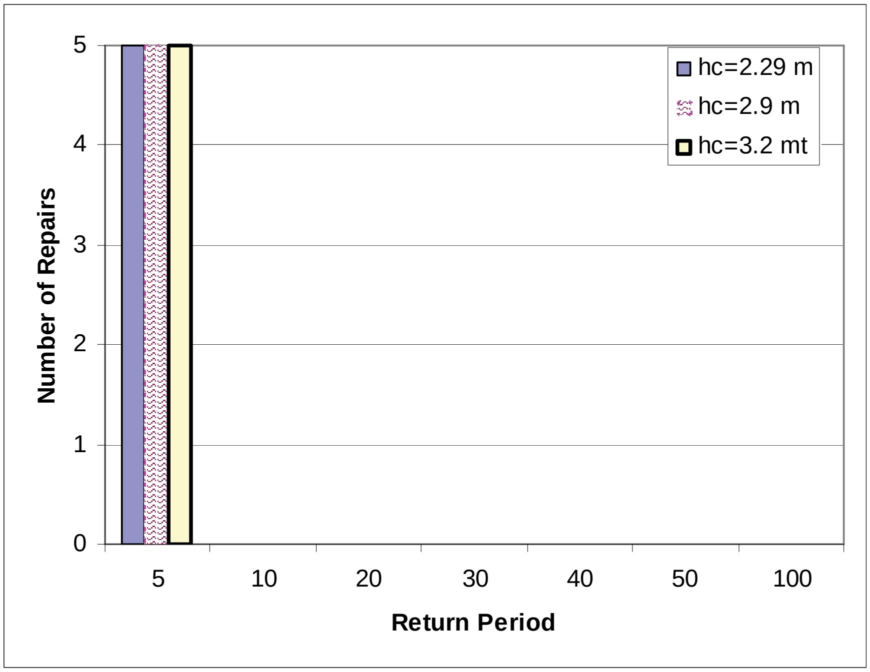


Figure 85. Total number of repairs as function of return period at sta 37 for several crest heights. Crests were unarmored

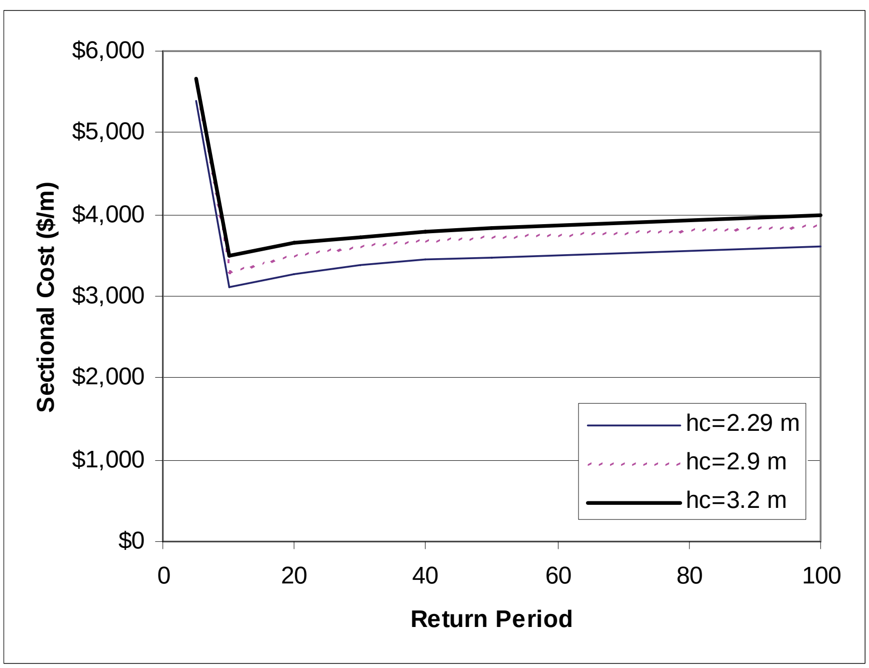


Figure 86. Total cross-sectional cost as function of return period at sta 37 for several crest heights. Crests were heavily armored

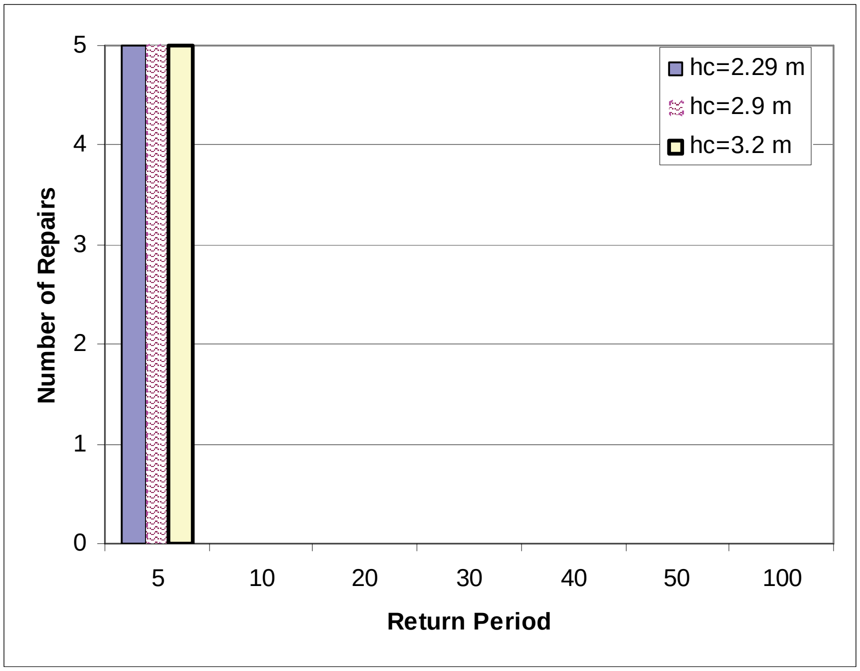


Figure 87. Total number of repairs as function of return period at sta 37 for several crest heights. Crests were heavily armored

Figures 88-91 show *Sectional Cost* and *Number of Repairs* for sta 39. The figures are similar to those shown for sta 33 and 37. The section with the unarmored crest increases in cost from the 20-year return period. The least-cost unarmored crest is the lowest at 2.9 m (9.5 ft). Figure 89 shows that all return-period unarmored-crest designs past 5 years require two repairs over the 50-year life. The armored crest section at sta 39, shown in Figures 90 and 91, is more efficient than the unarmored crest for this station. The least-cost armored-crest section has a crest height of 3.2 m (10.5 ft) and corresponds to the 50-year return period design. Figure 91 shows that the armored-crest design requires no repairs over the 50-year life cycle. As with sta 33, the sta 39 least-cost section has an armored crest. The least-cost armored and unarmored sections are nearly identical in cost. However, the larger return period design of the armored crest would have a higher degree of reliability and would therefore be superior.

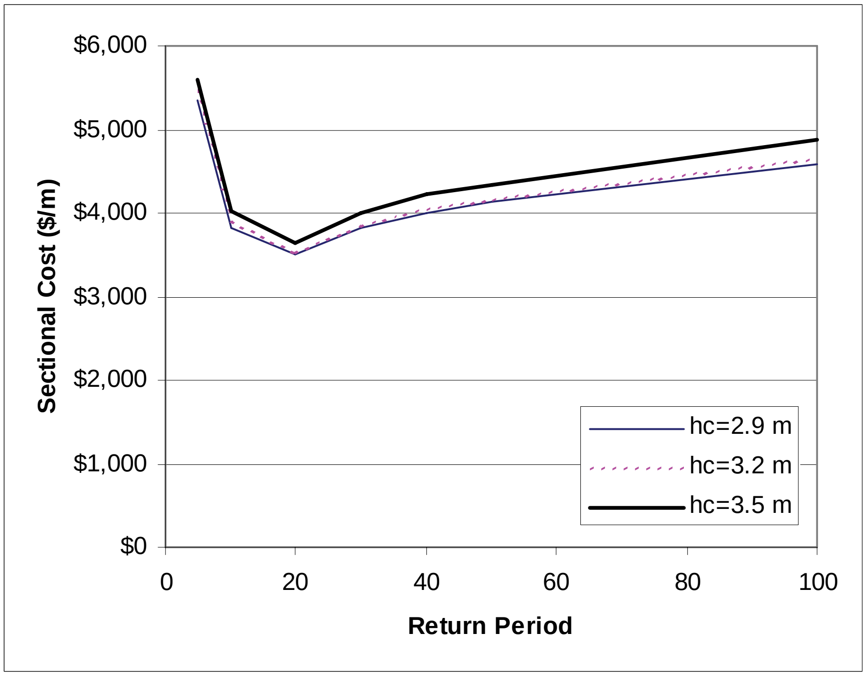


Figure 88. Total cross-sectional cost as function of return period at sta 39 for several crest heights. Crests were unarmored

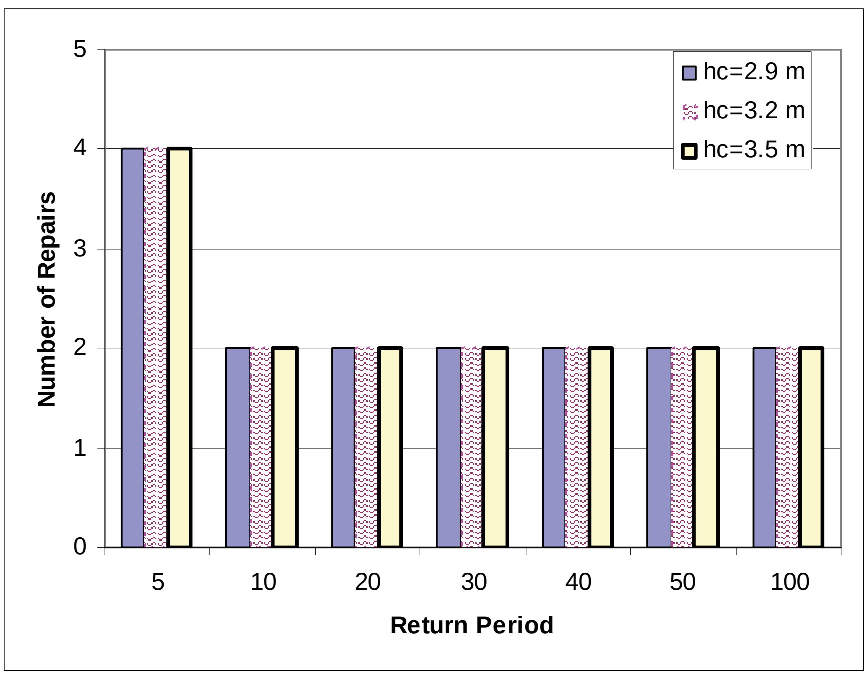


Figure 89. Total number of repairs as function of return period at sta 39 for several crest heights. Crests were unarmored

Figure 90. Total cross-sectional cost as function of return period at sta 39 for several crest heights. Crests were heavily armored

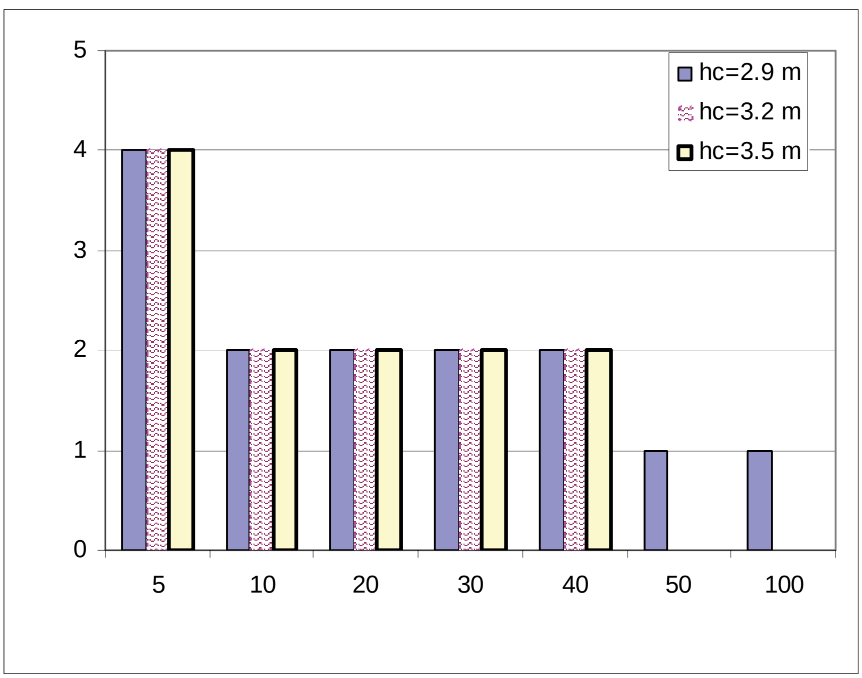


Figure 91. Total number of repairs as function of return period at sta 39 for several crest heights. Crests were heavily armored

The upland cell analysis consisted of analyzing the compound slope runup using Equations 9-17 in Chapter 6. The potential upland cells correspond to sta 37 and 38. As was shown above, these stations show no overtopping breaches for return periods greater than 5 years, regardless of crest height. The stations also show no runup on the upland cells for the higher return periods. As such, there is no need to armor the upland cell slopes as long as the crest height of the lower structure is maintained at a reasonably high level of greater than 2.29 m (7.5 ft) mllw.

Summary and Recommendations

From the results of the preliminary and ELS investigations summarized above a few conclusions can be made:

a. The least-cost solution that provides a reasonable level of protection for the reach represented by sta 33 and 34 (from the southwest start of expansion to the northwest entrance) is the 40-year return period design with an armored crest. This design can be expected to have at least three repairs during the 50-year life cycle. There is little apparent increase in cross-sectional cost in going to larger stone, given that stone unit costs and equipment costs do not increase with increasing stone size. However, this is likely not the case. As such, the recommended design for the western side of the expansion is based on sta 34 with the 40-year return period cross section, an armored crest at 2.9 m (9.5 ft), and a primary slope of 1V:3H.

b. Based on the preliminary design, the northern reach represented by sta 35 and 36 is between the sheltered eastern side and the exposed western side. The required stone sizes are between the two. The water level exposure is low, similar to the eastern side. The recommended design for this reach is based on sta 36 with the 40-year return period cross section, an armored crest at 2.9 m (9.5 ft), and a primary slope of 1V:3H.

c. The least-cost solution that provides a reasonable level of protection for the reach represented by sta 37 and 38 is based on the sta 37 cross section. The least-cost alternative corresponds to the 10-year return period design with an armored crest. The least-cost crest height is the lowest at 2.29 m (7.5 ft).

d. The least-cost solution that provides a reasonable level of protection for the reach represented by sta 39 corresponds to the 50-year return period design with an armored crest at 3.2 m (10.5 ft).

All of the following designs have a primary armor seaside slope of 1V:3H and a toe seaside slope of 1V:2H. The above analysis resulted in the following optimal sections.

Western reach, sta 33 and 34

a. Armor stone weight: 1.04 tonne (2,300 lb).

b. Primary underlayer stone weight: 0.10 tonne (230 lb).

c. Crest: h_c = 2.9 m (9.5 ft) armored.

d. Toe stone weight: 0.54 tonne (1,200 lb).

e. Toe underlayer weight: 0.05 tonne (120 lb).

Northern reach, sta 35 and 36

a. Armor stone weight: 0.29 tonne (650 lb).

b. Primary underlayer stone weight: 0.03 tonne (65 lb).

c. Crest: h_c = 2.9 m (9.5 ft) armored.

d. Toe stone weight: 0.15 tonne (330 lb).

e. Toe underlayer weight: 0.02 tonne (33 lb).

Eastern reach, sta 37 and 38

a. Armor stone weight: 0.05 tonne (120 lb).

b. Primary underlayer stone weight: 0.005 tonne (12 lb).

c. Crest: h_c = 2.29 m (7.5 ft) armored.

d. Toe stone weight: 0.027 tonne (60 lb).

e. Toe underlayer weight: 0.003 tonne (6 lb).

Southeastern reach, sta 39

a. Armor stone weight: 0.29 tonne (650 lb).

b. Primary underlayer stone weight: 0.03 tonne (65 lb).

c. Crest: h_c = 3.2 m (10.5 ft) armored.

d. Toe stone weight: 0.135 tonne (300 lb).

e. Toe underlayer weight: 0.01 tonne (35 lb).

The optimal design for sta 37 and 38 requires some additional review. As stated previously, the 10-year return period section was the least cost. However, Figures 82 and 83 showed that the costs were not a strong function of return period. There is little cost penalty in using a more reliable design with larger stone. A return period of 40 years would be more consistent with the other sections. Also, if conditions change, such as a sea level rise, or if there is stone breakage or poor construction over a reach, then it will be desirable to have a stronger design. Therefore, it is recommended that the design return period for the eastern reach be increased to 40 years. The design armor stone size would be 270 lbs.

The previous designs overlap somewhat. As such, several sections can be combined to reduce the number of stone classes and simplify the design. One possibility is to combine sta 35-39. However, this may not be advantageous because much smaller stone is required for sta 37 and 38. It is recommended that three different cross sections be used. The primary armor stone size on sta 37 and 38 is increased below to 330 lb to decrease the number of stone classes. Because the costs for the various crest heights are virtually the same, it may be useful to make the entire structure one crest height. In this case, the crest height of 2.9 m (9.5 ft) is recommended. The final design return period was 40 years for all stations except 39, which was 50 years. The final recommended sections are as follows:

Western reach, sta 33 and 34

a. Armor stone weight: 1.04 tonne (2300 lb).

b. Primary underlayer stone weight: 0.10 tonne (230 lb).

c. Crest: h_c = 2.9 m (9.5 ft) armored.

d. Toe stone weight: 0.54 tonne (1200 lb).

e. Toe underlayer weight: 0.054 tonne (120 lb).

Northern and southeastern reaches, sta 35, 36, and 39

a. Armor stone weight: 0.29 tonne (650 lb).

b. Primary underlayer stone weight: 0.03 tonne (65 lb).

c. Crest: h_c = 2.9 m (9.5 ft) armored.

d. Toe stone weight: 0.15 tonne (330 lb).

e. Toe underlayer weight: 0.015 tonne (33 lb).

Eastern reach, sta 37 and 38

a. Armor stone weight: 0.15 tonne (330 lb).

b. Primary underlayer stone weight: 0.015 tonne (33 lb).

c. Crest: h_c = 2.9 m (9.5 ft) armored.

d. Toe stone weight: 0.054 tonne (120 lb).

e. Toe underlayer weight: 0.005 tonne (12 lb).

National Marine Fisheries Service Offshore Breakwater Alternative

The offshore breakwater alternative proposed by the National Marine Fisheries Service (NMFS) is shown in Figure 92. This alternative replaces the original revetment with a series of segmented offshore breakwaters. The fill in the cell enclosed by the revetment and segmented breakwater in Figure 92 is replaced with a relatively calm embayment. The intent of this alternative is to provide increased fish habitat. A shoreline revetment is parallel to the original channel that runs generally north-south in the middle of the island. The analysis summarized here provides an initial optimization of the segmented offshore breakwater and the revetment using only historical wave conditions.

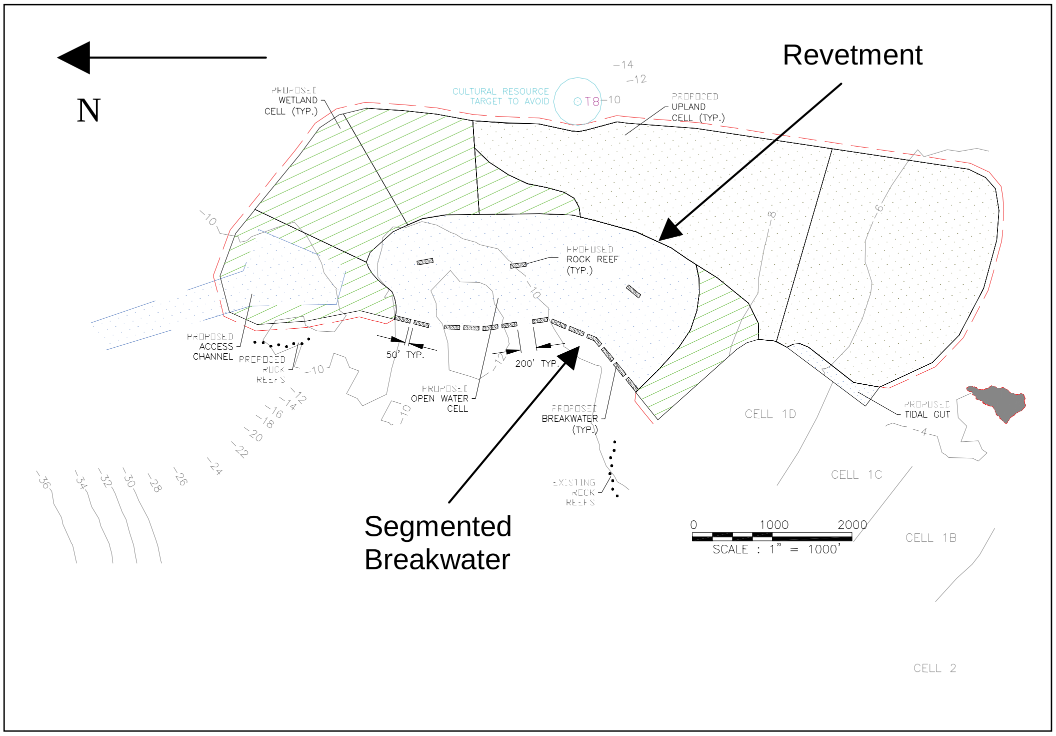


Figure 92. NMFS segmented offshore breakwater and revetment alternative

Offshore breakwater analysis

The first step in the analysis required computing the optimal cross section for the offshore breakwaters. Station 33 wave and water level data were used for the representative section. As part of this effort, a new program called LC_COST_BW was developed for breakwater analysis. This new program included low-crested or slightly submerged structure options. The armor stone sizes were decreased using Equation 43 to account for the low crest. If the crest was submerged, the armor stone size was computed using Equations 39-42. Wave transmission over the structure was computed according to Equations 21 and 22.

Input parameters for this analysis are summarized in Table 37. The toe stone was assumed to be the same size as the armor. For this effort, the structure was assumed to be configured as shown in Figure 93. The structure crest was three stones wide. The section included a traditional section with two-stone-thick armor and filter layers. For the final section, filter material may replace the core so there are only two stone classes. For all analyses, it was assumed that the structure was placed on a geotextile to prevent fines from leaching up through the structure.

Table 37
Input Parameter Values for NMFS Offshore Breakwater

Parameter	Variable	Value
Permeability	P	0.4
Porosity	Por	0.38
Stone specific gravity	S_r	2.578
Stone density	ρ_r	2.644 tonne/cu m (165 pcf)
Minor repair limit	S_M	8
Breach repair limit	S_B	18
Minor repair time limit	-	180 days
Breach repair time limit	–	120 days
Roughness parameter	γ_b	0.55
Crest width	-	$3D_{n50}$
Toe berm height	d_B	D_{toe}
Toe berm seaward slope	cot φ	N/A
Toe berm leeward slope	cot β	N/A
Toe berm crest width	-	$2D_{toe}$
Toe armor thickness	-	D_{toe}
Allowable main armor damage	S	2.0
Allowable toe damage	N_{od}	2.0
Number of waves for zero damage	N_z	7000
Inflation or escalation rate	I	0.03
Interest rate	R	0.05375
Economic life	N	50 years
Armor material unit cost	-	$56/tonne ($50.4/ton)
Filter material unit cost	-	$39/tonne ($35.1/ton)
Bedding material unit cost	-	$44/tonne ($39.6/ton)
Quarry-run material unit cost	-	$44/tonne ($39.6/ton)
Geotechnical material unit cost	-	$4.78/sq m ($0.44/sq ft)
Lag before initial construction	Lag	0 years
Fixed first cost	FFC/L_s	$500/m ($152/ft)
Fixed repair cost	RFC/L_r	$2,500/m ($7,62/ft)
Structure slopes	Cot α	Varied
Ratio of repair length to section	L_r/L_s	0.3

The maximum surge levels in the historical record near ADCIRC save station locations 1 and 2 (Figure 19), listed in Table 15, occurred during Hurricane Hazel. The maximum surge along this reach of Poplar Island was 2.048 m (6.72 ft) msl, or 2.278 m (7.47ft) mllw. This water level dictates the minimum height of the crest for the shoreline revetment because the structure would likely be undermined if flooding due to storm surge overtopped the structure. In addition, there would likely be significant damage to the ecosystem if significant flooding occurred. However, no such restriction has been placed on the crest height for the offshore breakwaters. In this investigation, an optimum combination of crest height for the offshore structures and armor size for the shoreline revetment is sought that will minimize costs and maintain functionality of the embayment.

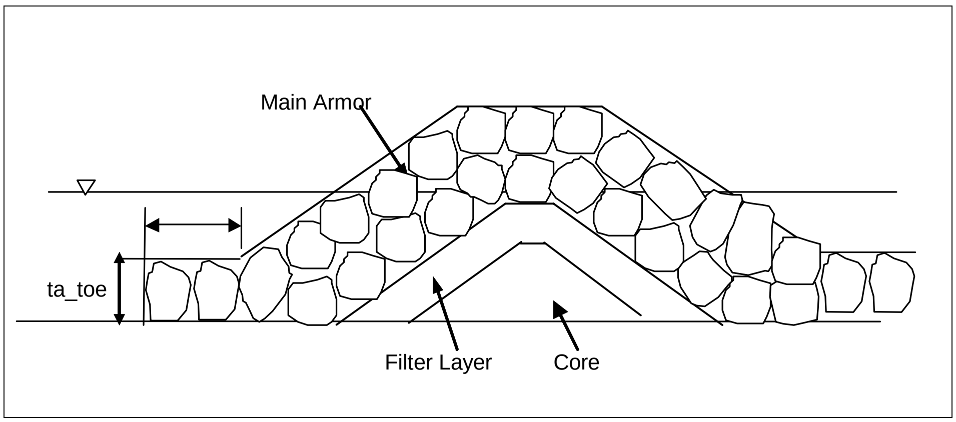


Figure 93. Typical section of offshore structure

Initial results for the offshore structure are plotted in Figure 94. Summary results are listed in Tables 38-41. Figure 94 shows that the least-cost alternative is the one with the smallest cross section and the largest armor stone. The steep slope alternatives with seaward and leeward side slopes of 1V:1.5H are the least expensive. Of these, the lowest crest is the least expensive. For the least-cost alternative with a crest height of 1.83 m (6 ft) mllw, the largest stone with a return period of 100 years is the least expensive. There is one minor damage repair and no breaches for return periods of 35-100 years. Although the larger stone is least expensive, it has been assumed that this stone has the same unit cost as the smaller stone, which may not be the case. So, the lower return period of 45 years is selected for design. For the next higher crest height of 2.13 m (7 ft) mllw, the least-cost alternative is the 40-year return period, and there is no repair for return periods of 40 years or greater.

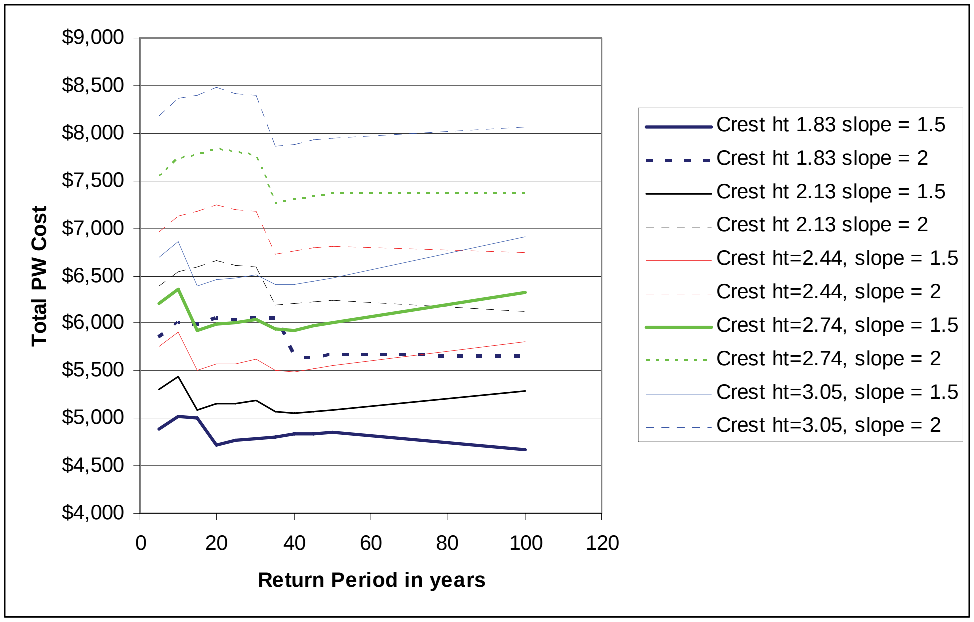


Figure 94. NMFS offshore breakwater analysis results. Total present-worth cost versus return period

Table 38
First Cost per Running Meter for Offshore Breakwater Crest Height of 1.83 m (6 ft) mllw and Structure Slope of 1V:1.5H

RP	Cost of Armor	Cost of Filter Layer	Cost of Core	Cost of Toe Armor	Cost of Geotextile	Fixed Cost	Total Cost
5	$1,309	$367	$1,806	$28	$43	$500	$4,053
10	$1,581	$429	$1,656	$42	$47	$500	$4,254
15	$1,701	$ 454	$1,591	$48	$48	$500	$4,342
20	$1,800	$ 474	$1,537	$54	$50	$500	$4,415
25	$1,868	$487	$1,501	$59	$51	$500	$4,465
30	$1,906	$494	$1,481	$61	$51	$500	$4,494
35	$1,979	$508	$1,442	$66	$52	$500	$4,548
40	$2,033	$518	$1,414	$70	$53	$500	$4,588
45	$2,028	$517	$1,417	$70	$53	$500	$4,584
50	$2,056	$522	$1,402	$72	$53	$500	$4,605
100	$2,090	$528	$1,384	$74	$54	$500	$4,630

Table 39
First Cost per Running Meter for Offshore Breakwater Crest Height of 2.13 m (7 ft) mllw and Structure Slope of 1V:1.5H

RP	Cost of Armor	Cost of Filter Layer	Cost of Core	Cost of Toe Armor	Cost of Geotextile	Fixed Cost	Total Cost
5	$1,387	$392	$2,065	$28	$47	$500	$4,419
10	$1,676	$459	$1,904	$42	$51	$500	$4,632
15	$1,888	$505	$1,789	$53	$54	$500	$4,788
20	$1,975	$522	$1,742	$58	$55	$500	$4,852
25	$2,042	$536	$1,706	$63	$56	$500	$4,902
30	$2,090	$545	$1,681	$66	$56	$500	$4,937
35	$2,213	$568	$1,616	$74	$58	$500	$5,028
40	$2,254	$576	$1,594	$77	$58	$500	$5,059
45	$2,261	$577	$1,590	$77	$58	$500	$5,064
50	$2,291	$582	$1,575	$80	$59	$500	$5,086
100	$2,557	$628	$1,437	$100	$62	$500	$5,284

Table 40
Summary of Offshore Breakwater Section for Crest Height of 1.83 m (6 ft) mllw and Structure Slope of 1V:1.5H

RP	Armor Stone D_{n50}	Armor Stone W_{50}	Filter Layer Stone D_{n50}	Filter Layer Stone W_{50}	Toe Stone D_{n50}	Toe Stone W_{50}	Present Worth First Cost/m	Present Worth Repair Cost/m	Present Worth Total Cost/m
5	0.39 m (1.29 ft)	1,559 N (353 lb)	0.18 m (0.60 ft)	157 N (36 lb)	0.39 m (1.29 ft)	1,559 N (353 lb)	$4,053	$828	$4,881
10	0.48 m (1.56 ft)	2,795 N (633 lb)	0.22 m (0.73 ft)	275 N (62 lb)	0.48 m (1.56 ft)	2,795 N (633 lb)	$4,254	$768	$5,022
15	0.51 m (1.68 ft)	3,501 N (793 lb)	0.24 m (0.78 ft)	353 N (80 lb)	0.51 m (1.68 ft)	3,501 N (793 lb)	$4,342	$653	$4,995
20	0.54 m (1.78 ft)	4,178 N (947 lb)	0.25 m (0.83 ft)	422 N (96 lb)	0.54 m (1.78 ft)	4,178 N (947 lb)	$4,415	$300	$4,716
25	0.57 m (1.85 ft)	4,688 N (1062 lb)	0.26 m (0.86 ft)	471 N (107 lb)	0.57 m (1.85 ft)	4,688 N (1,062 lb)	$4,465	$299	$4,764
30	0.58 m (1.89 ft)	4,992 N (1,131 lb)	0.27 m (0.88 ft)	500 N (113 lb)	0.58 m (1.89 ft)	4,992 N (1,131 lb)	$4,494	$298	$4,791
35	0.60 m (1.97 ft)	5,619 N (1,273 lb)	0.28 m (0.92 ft)	559 N (127 lb)	0.60 m (1.97 ft)	5,619 N (1,273 lb)	$4,548	$252	$4,800
40	0.62 m (2.03 ft)	6,110 N (1,384 lb)	0.29 m (0.94 ft)	608 N (138 lb)	0.62 m (2.03 ft)	6,110 N (1,384 lb)	$4,588	$251	$4,840
45	0.62 m (2.02 ft)	6,061 N (1,373 lb)	0.29 m (0.94 ft)	608 N (138 lb)	0.62 m (2.02 ft)	6061 N (1,373 lb)	$4,584	$251	$4,836
50	0.63 m (2.05 ft)	6,335 N (1,436 lb)	0.29 m (0.95 ft)	637 N (144 lb)	0.63 m (2.05 ft)	6,335 N (1,436 lb)	$4,605	$251	$4,856
100	0.64 m (2.09 ft)	6,659 N (1,509 lb)	0.30 m (0.97 ft)	667 N (151 lb)	0.64 m (2.09 ft)	6,659 N (1,509 lb)	$4,630	$44	$4,675

Table 41
Summary of Offshore Breakwater Section for Crest Height of 2.13 m (7 ft) mllw and Structure Slope of 1V:1.5H

RP	Armor Stone D_{n50}	Armor Stone W_{50}	Filter Layer Stone D_{n50}	Filter Layer Stone W_{50}	Toe Stone D_{n50}	Toe Stone W_{50}	Present Worth First Cost/m	Present Worth Repair Cost/m	Present Worth Total Cost/m
5	0.39 m (1.29 ft)	1,559 N (353 lb)	0.18 m (0.60 ft)	157 N (36 lb)	0.39 m (1.29 ft)	1,559 N (353 lb)	$4,419	$878	$5,298
10	0.48 m (1.56 ft)	2,795 N (633 lb)	0.22 m (0.73 ft)	275 N (62 lb)	0.48 m (1.56 ft)	2,795 N (633 lb)	$4,632	$811	$5,443
15	0.54 m (1.77 ft)	4,040 N (916 lb)	0.25 m (0.82 ft)	402 N (91 lb)	0.54 m (1.77 ft)	4,040 N (916 lb)	$4,788	$301	$5,089
20	0.56 m (1.85 ft)	4,648 N (1,053 lb)	0.26 m (0.86 ft)	461 N (104 lb)	0.56 m (1.85 ft)	4,648 N (1,053 lb)	$4,852	$299	$5,151
25	0.58 m (1.92 ft)	5,158 N (1,169 lb)	0.27 m (0.89 ft)	520 N (118 lb)	0.58 m (1.92 ft)	5,158 N (1,169 lb)	$4,902	$253	$5,155
30	0.60 m (1.96 ft)	5,541 N (1,256 lb)	0.28 m (0.91 ft)	559 N (127 lb)	0.60 m (1.96 ft)	5,541 N (1,256 lb)	$4,937	$252	$5,189
35	0.64 m (2.08 ft)	6,629 N (1,502 lb)	0.30 m (0.97 ft)	667 N (151 lb)	0.64 m (2.08 ft)	6,629 N (1,502 lb)	$5,028	$44	$5,073
40	0.65 m (2.12 ft)	7,022 N (1,591 lb)	0.30 m (0.99 ft)	706 N (160 lb)	0.65 m (2.12 ft)	7,022 N (1,591 lb)	$5,059	$0	$5,059
45	0.65 m (2.13 ft)	7,090 N (1,607 lb)	0.30 m (0.99 ft)	706 N (160 lb)	0.65 m (2.13 ft)	7,090 N (1,607 lb)	$5,064	$0	$5,064
50	0.66 m (2.16 ft)	7,384 N (1,673 lb)	0.31 m (1.00 ft)	735 N (167 lb)	0.66 m (2.16 ft)	7,384 N (1,673 lb)	$5,086	$0	$5,086
100	0.74 m (2.42 ft)	10,434 N (2,364 lb)	0.34 m (1.13 ft)	1040 N (236 lb)	0.74 m (2.42 ft)	10,434 N (2,364 lb)	$5,284	$0	$5,284

Revetment analysis

The shoreline revetment analysis is similar to that described previously. The structure parameters are those listed in Table 32, except the *Lag* is set to zero (i.e., the economic calculation assumes that year 0 is this year). The crest height is fixed at 2.13 m (7 ft). The representative cross section is shown in Figure 58. An unarmored crest with a crest width of 7.62 m (25 ft) is assumed. The program LC_COST_REV is used to determine the optimal cross section given the transmitted wave height.

For the design of the revetment, the transmitted wave height past the offshore breakwater must be determined. The wave transmission will occur because of wave overtopping, wave diffraction through the gaps, and wave transmission by porous flow through the structures. For this preliminary analysis, it was assumed that wave transmission due to porous flow was negligible. Wave transmission due to overtopping depends on the offshore structure crest height and the incident wave height, wave length, and water level. Wave transmission due to diffraction depends on the gap width and the wave period. The overtopping transmission was assumed to be constant along the length of the revetment, while that due to diffraction will vary with location.

Overtopping transmission $C_t = (H_{mo})_{tOT}/(H_{mo})_i$ was computed using Equations 21 and 22 within program LC_COST_BW. In this case, the wave transmission is reported only for specific return-period wave and water level conditions. Wave transmission due to diffraction $K_d = (H_{mo})_{td}/(H_{mo})_i$ was computed using irregular wave diffraction diagrams given in Goda (1985). Time and funding constraints limited the complexity of this analysis. Diffraction coefficients for all return periods were determined for each gap shown in Figure 92. Diffraction coefficients were determined at 30-m (100-ft) increments along the revetment for each gap. The shoreline grid origin was directly landward of the northern-most gap. The squared diffraction coefficients were summed along with the squared wave transmission coefficients at each grid point to get total transmitted wave energy at each grid point. Diffraction was approximated alongshore using the relation $K_d^2 = A\exp(-x/B/C)$, where B is the gap width and A and C are empirical best-fit coefficients. Here A ranged from 0.043 to 0.09 and C ranged from 5 to 7, both depending on return period. Figures 95 and 96 show the resulting diffraction coefficients alongshore for gap widths of 61 m (200 ft) and 15 m (50 ft), respectively, for varied return periods. Figure 97 shows the final diffraction coefficients alongshore.

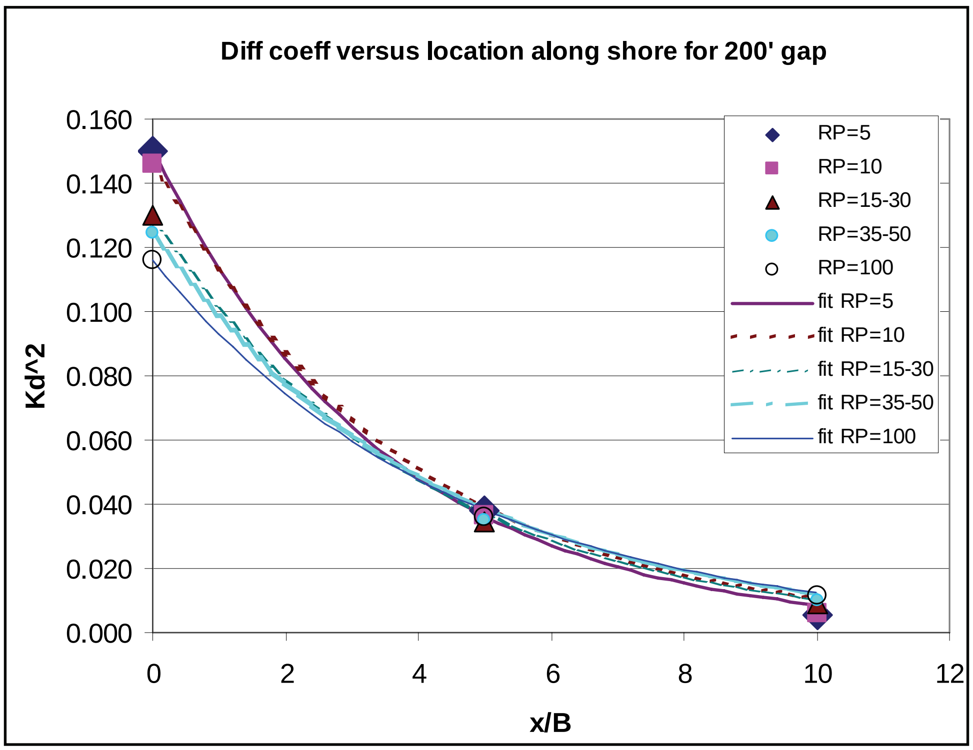


Figure 95. Squared diffraction coefficient as function of normalized distance along the revetment for 61-m (200-ft) gap width (where x is zero at northern end and B is gap width)

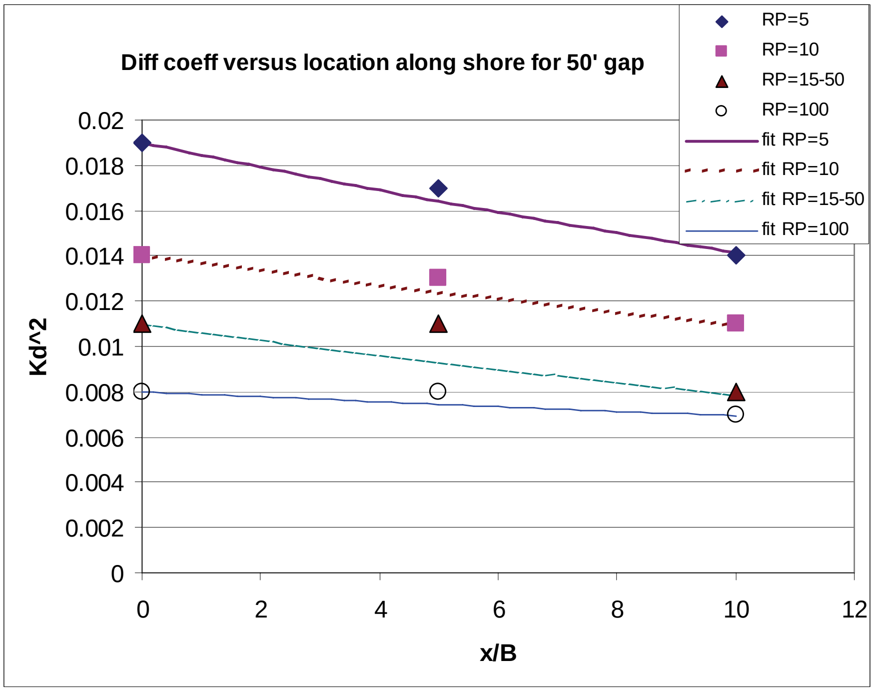


Figure 96. Squared diffraction coefficient as function of normalized distance along revetment for 15-m (50-ft) gap width (where *x* is zero at northern end and *B* is gap width)

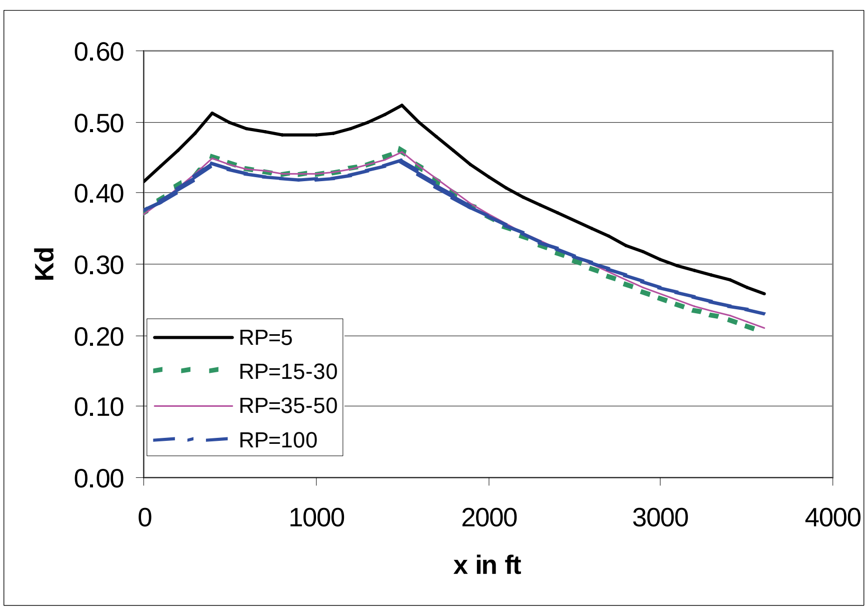


Figure 97. Diffraction coefficient alongshore from north end of revetment (x = 0) to south end [x = 1,097 m (3,600 ft)] for varied return periods (RP)

As shown in Figure 97, the diffraction coefficient does not vary all that much with return period. For design purposes, we are only interested in longer return periods. The curves for all return periods greater than 15 years align and can be considered to be a single curve. The total transmitted wave height was computed as $(H_{mo})_t = (H_{mo})_i K_T = (H_{mo})_i \sqrt{K_d^2 + C_t^2}$. Figures 98-100 show the total transmitted wave heights as a function of return period for several locations along the revetment.

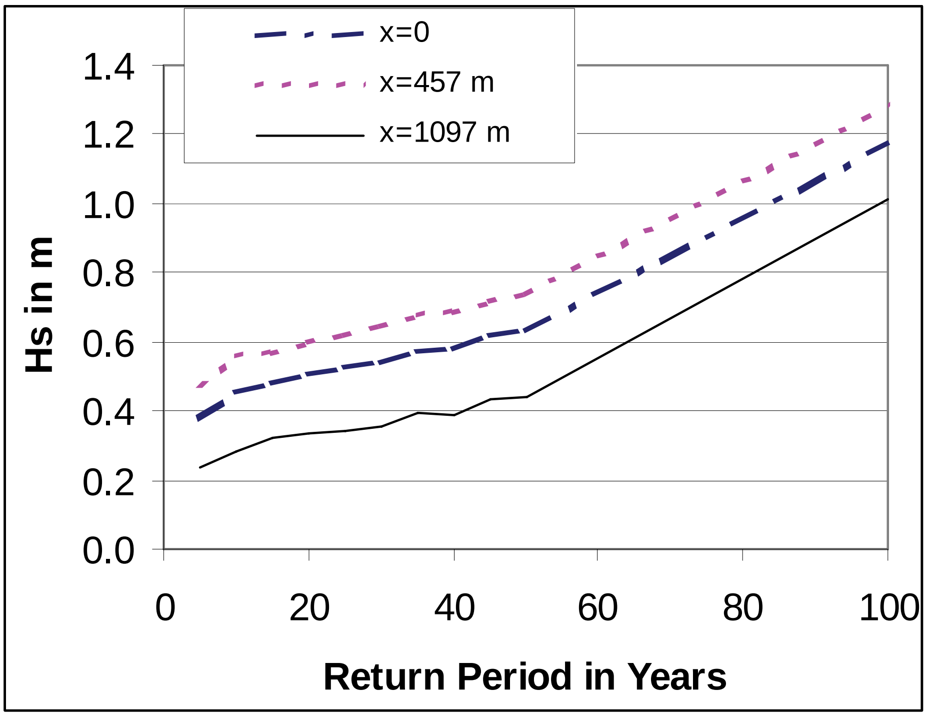


Figure 98. Significant wave height at three locations along revetment as function of return period for offshore breakwater crest height of 1.83 m (6 ft). *x* = 457 m (1,500 ft) corresponds to location landward of southern 61-m (200-ft) gap, while *x* = 1,097 m (3,600 ft) corresponds to southern end of revetment

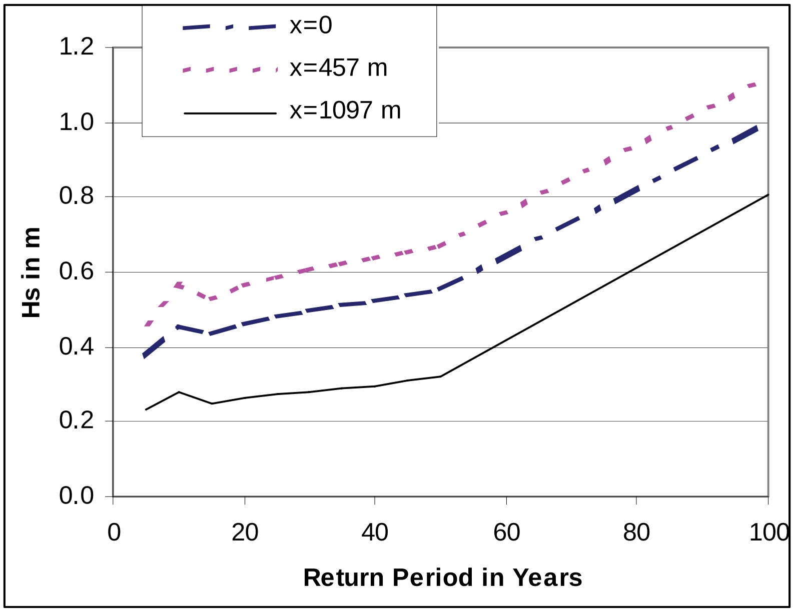


Figure 99. Significant wave height at three locations along revetment as function of return period for offshore breakwater crest height of 2.44 m (8 ft). *x* = 457 m (1,500 ft) corresponds to location landward of southern 61-m (200-ft) gap, while *x* = 1,097 m (3,600 ft) corresponds to southern end of revetment

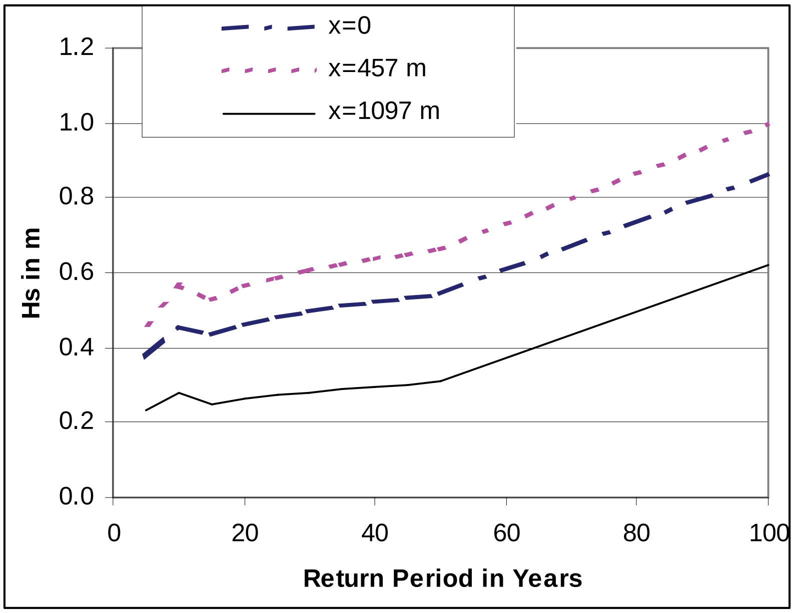


Figure 100. Significant wave height at three locations along revetment as function of return period for offshore breakwater crest height of 3.05 m (10 ft). *x* = 457 m (1,500 ft) corresponds to location landward of southern 61-m (200-ft) gap, while *x* = 1,097 m (3,600 ft) corresponds to southern end of revetment

The wave heights shown in Figures 98-100, along with the original return period wave periods and water levels for sta 33, were used to determine the revetment cross sections. The resulting section for the 1.83-m (6-ft) crest height offshore breakwater is summarized as a function of return period in Tables 42 and 43 for two locations along the revetment: (a) landward of the southern 61-m (200-ft) gap and (b) at the southern end of the revetment.

Table 42
First Cost per Running Meter for Revetment with Offshore Breakwater Crest Height of 1.83 m (6 ft) mllw at *x* = 457 m (1,500 ft)

RP	Cost of Armor	Cost of Filter Layer	Cost of Core	Cost of Toe Armor	Cost of Geotextile	Fixed Cost	Total Cost
5	$267	$1,563	$202	$365	$56	$500	$2,987
10	$324	$1,566	$202	$485	$56	$500	$3,167
15	$348	$1,562	$202	$555	$56	$500	$3,258
20	$360	$1,564	$202	$567	$56	$500	$3,283
25	$376	$1,567	$202	$587	$56	$500	$3,322
30	$385	$1,567	$202	$602	$56	$500	$3,346
35	$415	$1,571	$202	$638	$56	$500	$3,416
40	$415	$1,573	$202	$625	$56	$500	$3,406
45	$436	$1,570	$202	$680	$56	$500	$3,479
50	$441	$1,570	$202	$688	$56	$500	$3,491
100	$729	$1,443	$202	$1,564	$56	$500	$4,529

Table 43
First Cost per Running Meter for Revetment with Offshore Breakwater Crest Height of 1.83 m (6 ft) mllw at *x* = 1,097 m (3,600 ft)

RP	Cost of Armor	Cost of Filter Layer	Cost of Core	Cost of Toe Armor	Cost of Geotextile	Fixed Cost	Total Cost
5	$157	$1,540	$202	$219	$56	$500	$2,708
10	$194	$1,546	$202	$287	$56	$500	$2,820
15	$232	$1,551	$202	$362	$56	$500	$2,937
20	$235	$1,552	$202	$361	$56	$500	$2,941
25	$241	$1,554	$202	$366	$56	$500	$2,953
30	$251	$1,555	$202	$381	$56	$500	$2,979
35	$280	$1,561	$202	$416	$56	$500	$3,049
40	$281	$1,562	$202	$408	$56	$500	$3,043
45	$302	$1,563	$202	$454	$56	$500	$3,111
50	$307	$1,563	$202	$461	$56	$500	$3,124
100	$613	$1,493	$202	$1,256	$56	$500	$4,154

Tables 42-45 indicate that the minimum cost for revetment landward of the southern 61-m (200-ft) gap occurs for larger return periods. Because some secondary costs of repair are unknown and because repairs are generally not desirable, it is expected that even larger return periods will be more economical. Therefore, a return period of 45 years is selected for this preliminary design.

Table 44
Summary of Revetment Section with Offshore Breakwater Crest Height of 1.83 m (6 ft) mllw at *x* = 457 m (1,500 ft)

RP	Armor Stone D_{n50}	Armor Stone W_{50}	Filter Layer Stone D_{n50}	Filter Layer Stone W_{50}	Toe Stone D_{n50}	Toe Stone W_{50}	Present Worth First Cost/m	Present Worth Repair Cost/m	Present Worth Total Cost/m
5	0.25 m (0.81 ft)	392 N (89 lb)	0.12 m (0.38 ft)	39 N (9 lb)	0.24 m (0.80 ft)	373 N (84 lb)	$2,854	$3,858	$6,712
10	0.30 m (0.98 ft)	696 N (158 lb)	0.14 m (0.46 ft)	69 N (16 lb)	0.31 m (1.03 ft)	804 N (182 lb)	$3,025	$2,286	$5,311
15	0.32 m (1.06 ft)	863 N (196 lb)	0.15 m (0.49 ft)	88 N (20 lb)	0.35 m (1.16 ft)	1,147 N (260 lb)	$3,113	$2,055	$5,168
20	0.33 m (1.09 ft)	951 N (216 lb)	0.15 m (0.51 ft)	98 N (22 lb)	0.36 m (1.18 ft)	1,206 N (273 lb)	$3,137	$2,024	$5,160
25	0.35 m (1.14 ft)	1,089 N (247 lb)	0.16 m (0.53 ft)	108 N (24 lb)	0.37 m (1.22 ft)	1,324 N (300 lb)	$3,174	$1,947	$5,121
30	0.36 m (1.17 ft)	1,167 N (264 lb)	0.17 m (0.54 ft)	118 N (27 lb)	0.38 m (1.25 ft)	1,422 N (322 lb)	$3,197	$1,959	$5,156
35	0.38 m (1.26 ft)	1,461 N (331 lb)	0.18 m (0.58 ft)	147 N (33 lb)	0.40 m (1.31 ft)	1,648 N (373 lb)	$3,264	$1,995	$5,258
40	0.38 m (1.26 ft)	1,471 N (333 lb)	0.18 m (0.58 ft)	147 N (33 lb)	0.39 m (1.29 ft)	1,569 N (356 lb)	$3,254	$1,990	$5,244
45	0.40 m (1.32 ft)	1,697 N (384 lb)	0.19 m (0.61 ft)	167 N (38 lb)	0.42 m (1.38 ft)	1,942 N (440 lb)	$3,324	$2,027	$5,350
50	0.41 m (1.34 ft)	1,755 N (398 lb)	0.19 m (0.62 ft)	177 N (40 lb)	0.43 m (1.40 ft)	2,001 N (453 lb)	$3,335	$2,033	$5,368
100	0.67 m (2.21 ft)	7,934 N (1,798 lb)	0.31 m (1.03 ft)	794 N (180 lb)	0.83 m (2.73 ft)	14,985 N (3396 lb)	$4,327	$2,561	$6,888

Table 45
Summary of Revetment Section with Offshore Breakwater Crest Height of 1.83 m (6 ft) mllw at *x* = 1,097 m (3,600 ft)

RP	Armor Stone D_{n50}	Armor Stone W_{50}	Filter Layer Stone D_{n50}	Filter Layer Stone W_{50}	Toe Stone D_{n50}	Toe Stone W_{50}	Present Worth First Cost/m	Present Worth Repair Cost/m	Present Worth Total Cost/m
5	0.15 m (0.48 ft)	78 N (18 lb)	0.07 m (0.22 ft)	10 N (2 lb)	0.15 m (0.50 ft)	88 N (20 lb)	$2,587	$15,214	$17,801
10	0.18 m (0.59 ft)	147 N (33 lb)	0.08 m (0.27 ft)	20 N (4 lb)	0.20 m (0.64 ft)	196 N (44 lb)	$2,694	$9,208	$11,902
15	0.22 m (0.71 ft)	255 N (58 lb)	0.10 m (0.33 ft)	29 N (7 lb)	0.24 m (0.79 ft)	363 N (82 lb)	$2,806	$5,901	$8,707
20	0.22 m (0.71 ft)	265 N (60 lb)	0.10 m (0.33 ft)	29 N (7 lb)	0.24 m (0.79 ft)	363 N (82 lb)	$2,810	$5,106	$7,915
25	0.22 (0.73 ft)	284 N (64 lb)	0.10 m (0.34 ft)	29 N (7 lb)	0.24 m (0.80 ft)	373 N (84 lb)	$2,821	$4,615	$7,437
30	0.23 m (0.76 ft)	324 N (73 lb)	0.11 m (0.35 ft)	29 N (7 lb)	0.25 m (0.83 ft)	422 N (96 lb)	$2,846	$4,448	$7,294
35	0.26 m (0.85 ft)	451 N (102 lb)	0.12 m (0.39 ft)	49 N (11 lb)	0.27 m (0.90 ft)	530 N (120 lb)	$2,913	$3,549	$6,462
40	0.26 m (0.85 ft)	451 N (102 lb)	0.12 m (0.39 ft)	49 N (11 lb)	0.27 m (0.88 ft)	510 N (116 lb)	$2,907	$3,390	$6,298
45	0.28 m (0.92 ft)	569 N (129 lb)	0.13 m (0.43 ft)	59 N (13 lb)	0.30 m (0.97 ft)	677 N (153 lb)	$2,972	$2,260	$5,232
50	0.28 m (0.93 ft)	588 N (133 lb)	0.13 m (0.43 ft)	59 N (13 lb)	0.30 m (0.98 ft)	706 N (160 lb)	$2,984	$2,266	$5,250
100	0.57 m (1.86 ft)	4,717 N (1,069 lb)	0.26 m (0.86 ft)	471 N (107 lb)	0.70 m (2.30 ft)	8,924 N (2,022 lb)	$3,969	$2,371	$6,340

Recommendations

The recommended stone weights and layer thicknesses for both the offshore breakwater and the revetment section are as follows:

Offshore breakwater design at *x* = 457 m (1,500 ft)

a. Crest height: 1.83 m (6 ft).

b. Armor weight: 1,697 N (384 lb).

c. Armor thickness: 0.80 m (2.64 ft).

d. Filter layer weight: 167 N (38 lb).

e. Filter layer thickness: 0.38 m (1.22 ft).

f. Toe armor weight: 1,942 N (440 lb).

Revetment design

a. Crest height: 2.13 m (7 ft).

b. Armor weight: 1,697 N (384 lb).

c. Armor thickness: 0.80 m (2.64 ft).

d. Filter layer weight: 167 N (38 lb).

e. Filter layer thickness: 0.38 m (1.22 ft).

f. Toe armor weight: 1,942 N (440 lb).

8 Life-Cycle Simulation Results, James Island

This chapter describes the life-cycle structural optimization of the James Island revetment. Wave and water level results are presented in the following section. Methods used to develop these results are discussed in Chapter 5. Structure response and optimization are presented in the second section of this chapter. The methodology used to optimize the design of protective structures is given in Chapter 6. The methods used for James Island are identical to those used for Poplar Island, and the recommendations are similar.

Waves and Water Levels

The extremal H_s values for various return periods at each station are shown in Figure 101. The results are tabulated for each station in Appendix E to provide more background information. Stations with an open exposure toward the south and west experience the highest waves. These are also the stations most dominated by hurricanes. North- and east-facing sta 9-13 are less dominated by hurricanes. Return period H_s is relatively low at these stations, and the difference in H_s between the shortest and longest return periods is relatively small. Peak wave period and water level are shown as functions of return period for stations around James Island in Figures 102 and 103, respectively.

Structural Analysis

The structural analysis of James Island is composed of two primary parts: (a) preliminary analysis using only the historical waves and water levels, and (b) final design using the simulated waves and water levels. Both analyses use the program LC_COST_REV as the computation engine for the design sections and life-cycle response. For the preliminary design, the program is run for a large number of parametric permutations with only the historical wave and water level time series. The historical wave and water level time series was reordered so that the most recent years were first in the life cycle. In this way, the most recent years carried more weight. The final empirical life-cycle simulation (ELS) analysis using the empirically simulated waves and water levels is conducted for a narrow range of parametric permutations.

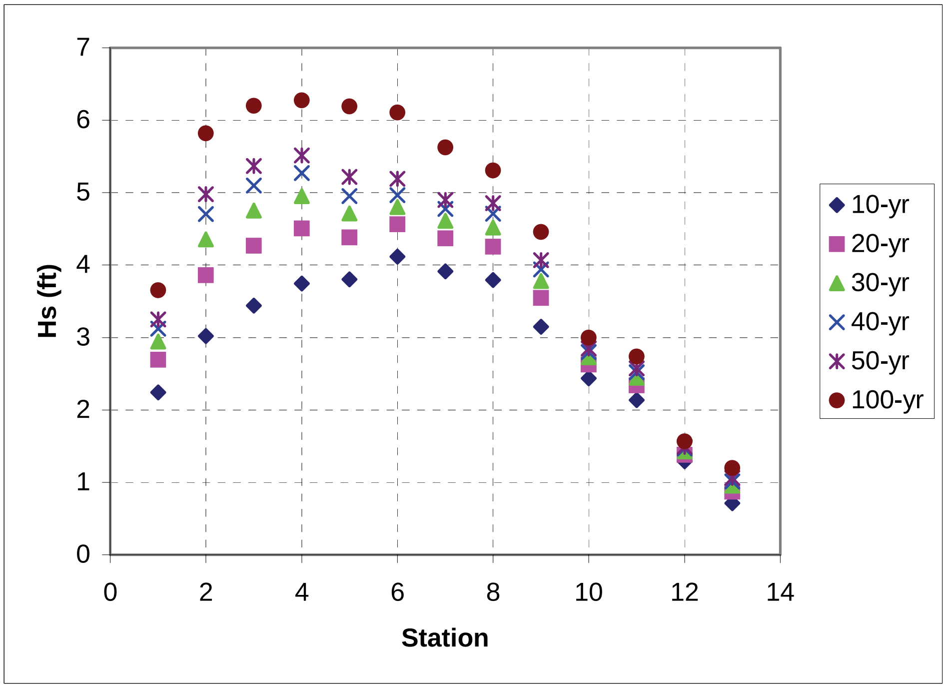


Figure 101. Return period H_s at nearshore stations, James Island

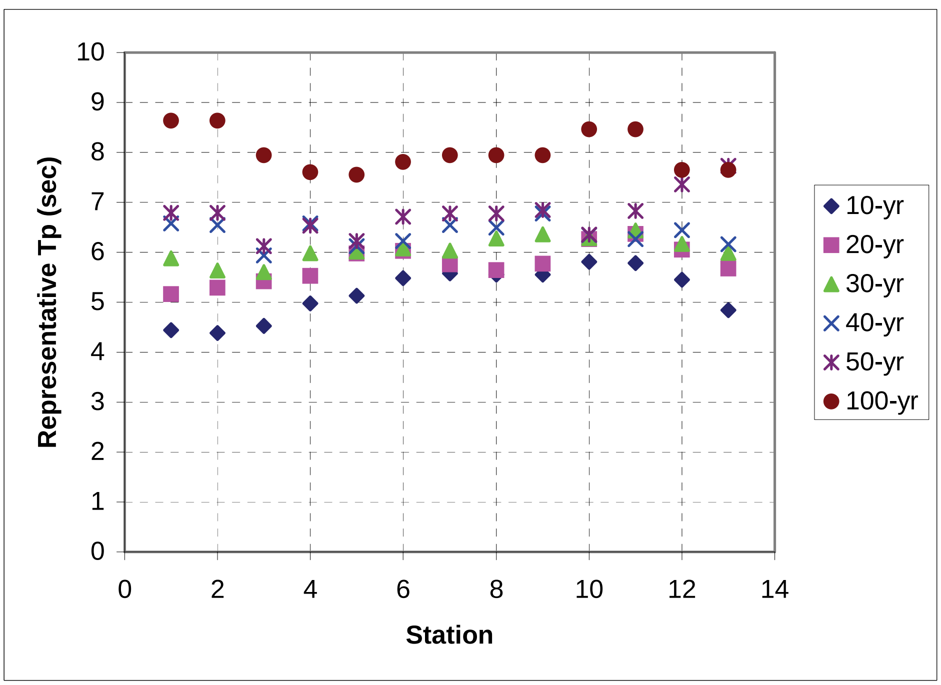


Figure 102. Return period T_p at nearshore stations, James Island

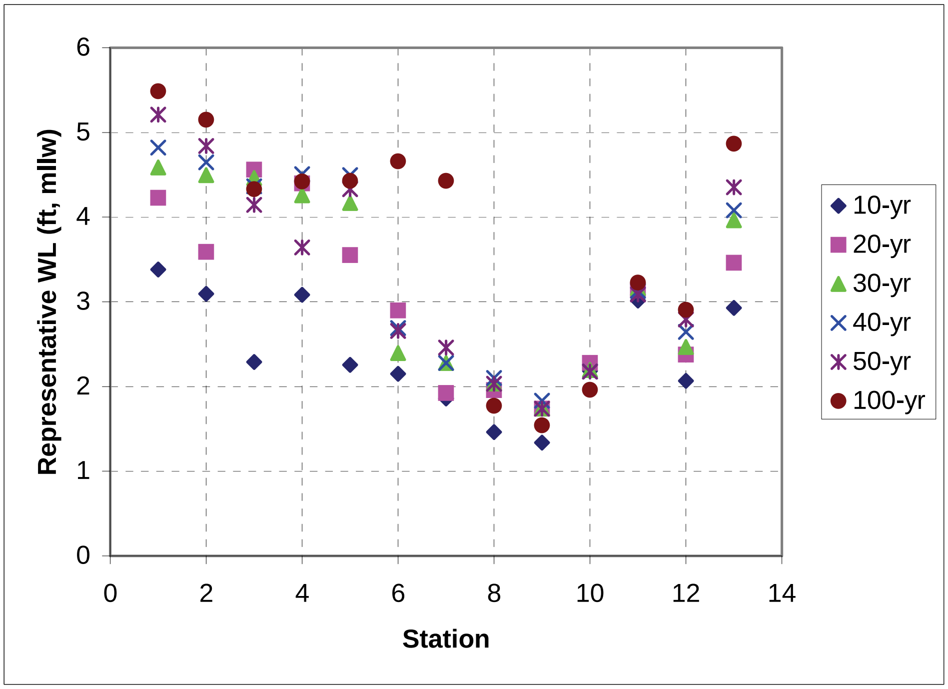


Figure 103. Return period water level at nearshore stations, James Island

The primary alternative had the general geometry of cross sections constructed for Poplar Island, as shown in Figure 58. This cross section has an unarmored crest. Figure 59 shows an armored crest alternative. The upland cell configuration shown in Figure 60 is proposed for the northern third of James Island. The layer thicknesses are assumed to be $2D_{a50}$ for armor, $2D_{u50}$ for the filter layer, 0.3 m (1 ft) for the bedding layer, 20 cm (8 in.) for the rock roadway, and $2D_{ta50}$ for the toe armor. Here $D_{a50} = (V_{50})^{1/3} = (W_{50}/\gamma_r)^{1/3}$ is the nominal diameter of the armor corresponding to the 50 percent exceedance level on the weight distribution curve. Similarly, D_{u50} is the filter layer 50 percent exceedance nominal diameter, and D_{ta50} is the toe armor 50 percent exceedance nominal diameter. The armored crest detail includes a single layer of armor across the crest with a full $2D_{u50}$-thick filter layer and a 0.3-m (1-ft) thick bedding layer. The three layers are tied back into the fill material at the lee side of the roadway extending down to mllw.

The filter layer thickness under the toe armor is determined by fixing the toe crest elevation at +0.3 m (1 ft) mllw and requiring the crest armor thickness to be $2D_{ta50}$. The filter layer is sized such that $W_{u50} = W_{50}/10$. The bedding material is assumed to be crushed gravel-sized material.

The analysis in LC_COST_REV contains two primary failure modes: armor stability and overtopping erosion of the crest. Toe damage is computed but the toe stability equation is not reliable because it often does not converge. Therefore, toe damage is not considered as a failure mode. At this time, there is no reliable toe damage progression model.

Overtopping rate limits of 0.05 cu m/sec/m (4.0 gal/sec/ft), corresponding to extensive damage on an unarmored crest; 0.2 cu m/sec/m (16.1 gal/sec/ft), corresponding to damage on a paved crest; and 1,000 cu m/sec/m (80,519 gal/sec/ft), corresponding to a fictitious damage limit on a heavily armored crest, were used in the optimization. The *Coastal Engineering Manual* and the *CIRIA Rock Manual* both contain equations for crest armor stability. However, the empirical equations provide nothing better than crude estimates of stable stone weight. All applicable equations and figures were investigated to determine stable crest armor requirements for James Island. Although estimates of stable weight varied by a factor of four or more between the different methods, three of the methods agreed to within roughly 20 percent and the average estimated stone weights were within roughly 10 percent of the primary structure armor weight. Therefore, in this study, we assume that the *heavily armored* crest is armored with a single layer of main armor. The crest armoring options are summarized as follows:

a. **Unarmored**. Gravel on geotextile, overtopping limit = 0.05 cu m/sec/m (4.0 gal/sec/ft). The consequence of exceeding the overtopping limit is structure breach.

b. **Paved**. Asphalt pavement, overtopping limit = 0.20 cu m/sec/m (16.1 gal/sec/ft). The consequence of exceeding the overtopping limit is structure breach.

c. **Heavily armored**. Single layer of main armor on filter layers, overtopping limit = 1,000 cu m/sec/m (80,519 gal/sec/ft). The overtopping limit will never be reached. The stone is sized for 2 percent displacement by count for return-period wave conditions.

Preliminary analysis using historical waves and water levels

The preliminary structural optimization for James Island for the historical wave climate is separated into two parts: optimization for least cost and optimization for fewest repairs.

Tables in Appendix E show the significant wave height, peak period, and depth as a function of return period from the extremal wave height analysis, discussed previously. As discussed in Chapter 5, the wave heights were computed from an extremal analysis, and the wave periods and water depths were bin-averaged for each wave height. In Appendix F, tables summarize the stable main armor, stable filter layer, and stable toe armor weights and nominal diameters as a function of return period for all design analysis stations of James Island. These stable armor weights were computed using Equations 30-34 and the extremal waves from Appendix E. The stable armor weights were computed using a seaward structure slope of cot α = 3.0, as specified for Poplar Island based on the requirement of a stable slope during construction.

The stone sizes in Appendix F based on the extremal waves were used to develop potential design cross sections. The armor, filter layer, and toe stone weights are summarized in Table 46 for a structure slope of cot α = 3.0. The corresponding layer thicknesses are summarized in Table 47. Tables 48 and 49 summarize the total sectional stone masses and sectional costs per unit length for armored and unarmored crests, respectively, for selected crest heights.

In the optimization, these sections were exposed to life cycles of either historical waves and water levels (preliminary analysis) or simulated and historical waves and water levels (final analysis) in order to calculate the life-cycle response of the structure. The fixed input parameters for the analysis are summarized in Table 28, and parameters that were varied are listed in Table 29 in Chapter 6. The mllw depths (Table 26) and extremal wave parameters (Appendix E) were constant for all simulations but unique for each station.

Table 46
Stable Stone Weights for Selected Stations and Several Return Periods

Station	Return Period					
	10 year	**20 year**	**30 year**	**40 year**	**50 year**	**100 year**
Main Armor Stable Stone Weight W_{a50}, N (lb)						
1	794 (178)	1,400 (315)	1,923 (432)	2,226 (501)	2,645 (595)	3,487 (784)
3	2,430 (546)	5,368 (1,207)	7,548 (1,697)	9,003 (2,024)	10,463 (2,352)	1,6393 (3,685)
5	3,570 (803)	5,932 (1,334)	7,826 (1,759)	8,736 (1,964)	9,837 (2211)	1,6798 (3,776)
7	3,782 (850)	5,152 (1,158)	6,442 (1,448)	7,013 (1,577)	7,111 (1599)	1,3182 (2,963)
8	3,426 (770)	4,969 (1,117)	6,169 (1,387)	6,504 (1,462)	7,311 (1644)	8,850 (1,990)
10	1,064 (239)	1,367 (307)	1,586 (357)	1,711 (385)	1,801 (405)	2,078 (467)
11	651 (146)	801 (180)	939 (211)	1,027 (231)	1,111 (250)	1,324 (298)
12	200 (45)	249 (56)	286 (64)	298 (67)	306 (69)	375 (84)
13	54 (12)	90 (20)	124 (28)	145 (33)	174 (39)	246 (55)
Underlayer Stable Stone Weight W_{u50}, N (lb)						
1	80 (18)	140 (31)	194 (44)	222 (50)	267 (60)	348 (78)
3	242 (54)	538 (121)	756 (170)	905 (204)	1,045 (235)	1,648 (370)
5	357 (80)	592 (133)	786 (177)	881 (198)	991 (223)	1,685 (379)
7	380 (85)	514 (116)	644 (145)	706 (159)	713 (160)	1,324 (298)
8	344 (77)	497 (112)	618 (139)	651 (146)	734 (165)	889 (200)
10	107 (24)	138 (31)	158 (35)	171 (38)	182 (41)	209 (47)
11	64 (14)	80 (18)	94 (21)	103 (23)	111 (25)	133 (30)
12	20 (4)	25 (6)	29 (6)	30 (7)	30 (7)	38 (8)
13	5 (1)	9 (2)	12 (3)	14 (3)	17 (4)	25 (6)
Toe Armor Stable Stone Weight W_{t50}, N (lb)						
1	1,673 (376)	2,801 (630)	3,406 (766)	3,847 (865)	4,664 (1,049)	5,286 (1,188)
3	4,047 (910)	1,0851 (2,439)	1,1114 (2,499)	10,938 (2,459)	10,335 (2,323)	11,793 (2,651)
5	4,969 (1,117)	1,0167 (2,286)	12,837 (2,886)	13,941 (3,134)	13,331 (2,997)	14,836 (3,335)
7	4,664 (1,049)	5,816 (1,308)	6,819 (1,533)	6,819 (1,533)	7,046 (1,584)	14,729 (3,311)
8	3,426 (770)	5,423 (1,219)	6,692 (1,504)	7,211 (1,621)	7,896 (1,775)	8,926 (2,007)
10	1,378 (310)	1,723 (387)	1,909 (429)	2,021 (454)	2,137 (480)	2,288 (514)
11	1,574 (354)	1,992 (448)	2,288 (514)	2,430 (546)	2,561 (576)	3,056 (687)
12	394 (89)	520 (117)	592 (133)	664 (149)	699 (157)	856 (193)
13	168 (38)	306 (69)	454 (102)	532 (120)	644 (145)	1,000 (225)

Table 47
Stone Weights and Layer Thicknesses for Selected Return Periods and Selected Stations

Station	Armor Weight, N (lb)	Armor Layer Thickness, m (ft)	Filter Layer Weight, N (lb)	Filter Layer Thickness, m (ft)	Toe Armor Weight, N (lb)	Toe Armor Layer Thickness, m (ft)
20-year Return Period						
1	1,400 (315)	0.75 (2.47)	140 (31)	0.35 (1.15)	1,260 (283)	0.73 (2.39)
3	5,368 (1,207)	1.18 (3.87)	538 (121)	0.55 (1.80)	3,326 (748)	1.01 (3.30)
5	5,932 (1,334)	1.22 (4.00)	592 (133)	0.57 (1.86)	3,591 (807)	1.03 (3.39)
7	5,152 (1,158)	1.16 (3.82)	514 (116)	0.54 (1.77)	3,209 (721)	0.99 (3.26)
8	4,969 (1,117)	1.15 (3.77)	497 (112)	0.53 (1.75)	3,094 (695)	0.98 (3.22)
10	1,367 (307)	0.75 (2.45)	138 (31)	0.35 (1.14)	948 (213)	0.66 (2.17)
11	801 (180)	0.63 (2.05)	80 (18)	0.29 (0.95)	618 (139)	0.57 (1.88)
12	249 (56)	0.42 (1.39)	25 (6)	0.20 (0.64)	209 (47)	0.40 (1.31)
13	90 (20)	0.30 (0.99)	9 (2)	0.14 (0.46)	85 (19)	0.30 (0.97)
30-year Return Period						
1	1,923 (432)	0.84 (2.75)	194 (44)	0.39 (1.28)	1,623 (365)	0.79 (2.60)
3	7,548 (1,697)	1.32 (4.34)	756 (170)	0.61 (2.01)	4,590 (1032)	1.12 (3.67)
5	7,826 (1,759)	1.34 (4.39)	786 (177)	0.62 (2.04)	4,664 (1049)	1.13 (3.69)
7	6,442 (1,448)	1.25 (4.11)	644 (145)	0.58 (1.91)	3,935 (885)	1.06 (3.49)
8	6,169 (1,387)	1.24 (4.06)	618 (139)	0.57 (1.88)	3,760 (845)	1.05 (3.44)
10	1,586 (357)	0.79 (2.58)	158 (35)	0.36 (1.19)	1,064 (239)	0.69 (2.26)
11	939 (211)	0.66 (2.17)	94 (21)	0.31 (1.00)	706 (159)	0.60 (1.97)
12	286 (64)	0.44 (1.46)	29 (6)	0.21 (0.68)	232 (52)	0.41 (1.36)
13	124 (28)	0.34 (1.10)	12 (3)	0.16 (0.51)	107 (24)	0.32 (1.05)
40-year Return Period						
1	2,226 (501)	0.88 (2.89)	222 (50)	0.41 (1.34)	1,868 (420)	0.83 (2.72)
3	9,003 (2,024)	1.40 (4.60)	905 (204)	0.65 (2.14)	5,450 (1,225)	1.19 (3.89)
5	8,736 (1,964)	1.39 (4.55)	881 (198)	0.65 (2.12)	5,232 (1,176)	1.17 (3.84)
7	7,013 (1,577)	1.29 (4.23)	706 (159)	0.60 (1.97)	4,278 (962)	1.09 (3.59)
8	6,504 (1,462)	1.26 (4.13)	651 (146)	0.58 (1.92)	3,980 (895)	1.07 (3.50)
10	1,711 (385)	0.81 (2.64)	171 (38)	0.37 (1.23)	1,140 (256)	0.70 (2.31)
11	1,027 (231)	0.68 (2.23)	103 (23)	0.32 (1.04)	764 (172)	0.62 (2.02)
12	298 (67)	0.45 (1.48)	30 (7)	0.21 (0.69)	246 (55)	0.42 (1.38)
13	145 (33)	0.35 (1.16)	14 (3)	0.16 (0.54)	124 (28)	0.34 (1.10)

Table 48
Total Stone Weight and Cost Per Unit Length of Structure for Selected Return Periods and Selected Stations for Armored Crest at 1.83 m (6 ft) mllw

Station	Armor Weight, kN (ton)	Filter Layer Weight, kN (ton)	Toe Armor Weight, kN (ton)	Armor Cost, $/m ($/ft)	Filter Layer Cost, $/m ($/ft)	Toe Armor Cost, $/m ($/ft)	Total Cost, $/m ($/ft)
20-year Return Period							
1	108.4 (12.3)	88.6 (10.0)	40.1 (4.5)	$619 ($189)	$352 ($107)	$229 ($70)	$1,992 ($607)
3	169.6 (19.2)	214.2 (24.3)	98.5 (11.2)	$968 ($295)	$852 ($260)	$563 ($172)	$3,175 ($968)
5	175.2 (19.9)	254.0 (28.8)	111.5 (12.6)	$1,001 ($305)	$1010 ($308)	$637 ($194)	$3,440 ($1,049)
7	167.3 (19.0)	249.7 (28.3)	105.9 (12.0)	$993 ($303)	$605 ($184)	$3,345 ($1,020)	$993 ($303)
8	165.2 (18.7)	339.9 (38.5)	123.0 (13.9)	$943 ($287)	$1352 ($412)	$702 ($214)	$3,789 ($1,155)
10	107.5 (12.2)	352.3 (39.9)	81.1 (9.2)	$614 ($187)	$1401 ($427)	$463 ($141)	$3,270 ($997)
11	90.0 (10.2)	187.6 (21.3)	50.1 (5.7)	$514 ($157)	$746 ($227)	$286 ($87)	$2,338 ($713)
12	60.9 (6.9)	170.8 (19.4)	32.7 (3.7)	$348 ($106)	$679 ($207)	$187 ($57)	$2,006 ($611)
13	43.5 (4.9)	143.6 (16.3)	21.9 (2.5)	$248 ($76)	$571 ($174)	$125 ($38)	$1,736 ($529)
30-year Return Period							
1	120.5 (13.7)	100.4 (11.4)	47.7 (5.4)	$688 ($210)	$399 ($122)	$272 ($83)	$2,152 ($656)
3	190.0 (21.5)	220.7 (25.0)	113.2 (12.8)	$1,085 ($331)	$878 ($268)	$647 ($197)	$3,401 ($1,037)
5	192.3 (21.8)	259.5 (29.4)	124.5 (14.1)	$1,098 ($335)	$1032 ($315)	$711 ($217)	$3,633 ($1,107)
7	180.4 (20.4)	254.1 (28.8)	115.5 (13.1)	$1,011 ($308)	$660 ($201)	$3,492 ($1,064)	$1,011 ($308)
8	177.7 (20.1)	344.3 (39.0)	133.2 (15.1)	$1,015 ($309)	$1,369 ($417)	$761 ($232)	$3,936 ($1,200)
10	112.9 (12.8)	354.8 (40.2)	84.9 (9.6)	$644 ($196)	$1,411 ($430)	$485 ($148)	$3,333 ($1,016)
11	95.0 (10.8)	189.9 (21.5)	52.9 (6.0)	$543 ($165)	$755 ($230)	$302 ($92)	$2,392 ($729)
12	63.7 (7.2)	172.3 (19.5)	34.1 (3.9)	$364 ($111)	$685 ($209)	$195 ($59)	$2,036 ($620)
13	48.4 (5.5)	146.4 (16.6)	24.0 (2.7)	$276 ($84)	$582 ($177)	$137 ($42)	$1,788 ($545)
40-year Return Period							
1	126.5 (14.3)	106.5 (12.1)	52.4 (5.9)	$722 ($220)	$423 ($129)	$299 ($91)	$2237 ($682)
3	201.6 (22.8)	224.0 (25.4)	122.2 (13.8)	$1,151 ($351)	$891 ($271)	$698 ($213)	$3,532 ($1,077)
5	199.6 (22.6)	261.5 (29.6)	130.8 (14.8)	$1,140 ($347)	$1,040 ($317)	$747 ($228)	$3,719 ($1,133)
7	185.6 (21.0)	255.7 (29.0)	119.8 (13.6)	$1,017 ($310)	$684 ($208)	$3,552 ($1,083)	$1,017 ($310)
8	180.8 (20.5)	345.1 (39.1)	136.4 (15.5)	$1,032 ($315)	$1,372 ($418)	$779 ($237)	$3,976 ($1,212)
10	115.8 (13.1)	356.1 (40.3)	87.2 (9.9)	$661 ($202)	$1,416 ($432)	$498 ($152)	$3,368 ($1,026)
11	97.8 (11.1)	191.2 (21.7)	54.4 (6.2)	$558 ($170)	$761 ($232)	$311 ($95)	$2,422 ($738)
12	64.8 (7.3)	172.9 (19.6)	34.7 (3.9)	$370 ($113)	$687 ($210)	$198 ($60)	$2,048 ($624)
13	51.0 (5.8)	147.8 (16.7)	25.2 (2.9)	$291 ($89)	$588 ($179)	$144 ($44)	$1,815 ($553)

Table 49
Total Stone Weight and Cost Per Unit Length of Structure for Selected Return Periods and Selected Stations for Unarmored Crest at Varied Levels: Station 1 at 2.59 m (8.5 ft), Station 2-9 at 3.05 m (10 ft) and Station 10-13 at 2.13 m (7 ft)

Station	Armor Weight, kN (ton)	Filter Layer Weight, kN (ton)	Toe Armor Weight, kN (ton)	Armor Cost, $/m ($/ft)	Filter Layer Cost, $/m ($/ft)	Toe Armor Cost, $/m ($/ft)	Total Cost, $/m ($/ft)
20-year Return Period							
1	100.6 (11.4)	69.2 (7.8)	40.1 (4.5)	$574 ($175)	$275 ($84)	$229 ($70)	$1,946 ($593)
3	185.0 (21.0)	197.8 (22.4)	98.5 (11.2)	$1,056 ($322)	$787 ($240)	$563 ($172)	$3,319 ($1,012)
5	191.1 (21.7)	237.2 (26.9)	111.5 (12.6)	$1,091 ($333)	$943 ($287)	$637 ($194)	$3,585 ($1,093)
7	182.5 (20.7)	233.6 (26.5)	105.9 (12.0)	$1,042 ($318)	$929 ($283)	$605 ($184)	$3,490 ($1,064)
8	180.1 (20.4)	324.0 (36.7)	123.0 (13.9)	$1,029 ($314)	$1,289 ($393)	$702 ($214)	$3,933 ($1,199)
10	82.3 (9.3)	324.1 (36.7)	81.1 (9.2)	$470 ($143)	$1,289 ($393)	$463 ($141)	$3,044 ($928)
11	68.9 (7.8)	163.9 (18.6)	50.1 (5.7)	$393 ($120)	$652 ($199)	$286 ($87)	$2,153 ($656)
12	46.6 (5.3)	154.8 (17.5)	32.7 (3.7)	$266 ($81)	$616 ($188)	$187 ($57)	$1,891 ($576)
13	33.3 (3.8)	132.2 (15.0)	21.9 (2.5)	$190 ($58)	$526 ($160)	$125 ($38)	$1,662 ($507)
30-year Return Period							
1	111.8 (12.7)	78.7 (8.9)	47.7 (5.4)	$638 ($195)	$313 ($95)	$272 ($83)	$2,092 ($638)
3	207.2 (23.5)	202.4 (22.9)	113.2 (12.8)	$1,183 ($361)	$805 ($245)	$647 ($197)	$3,549 ($1,082)
5	209.7 (23.8)	241.0 (27.3)	124.5 (14.1)	$1,198 ($365)	$959 ($292)	$711 ($217)	$3,781 ($1,152)
7	196.7 (22.3)	236.8 (26.8)	115.5 (13.1)	$1,123 ($342)	$942 ($287)	$660 ($201)	$3,638 ($1,109)
8	193.8 (22.0)	327.2 (37.1)	133.2 (15.1)	$1,106 ($337)	$1,301 ($397)	$761 ($232)	$4,082 ($1,244)
10	86.4 (9.8)	325.2 (36.8)	84.9 (9.6)	$493 ($150)	$1,293 ($394)	$485 ($148)	$3,093 ($943)
11	72.7 (8.2)	165.0 (18.7)	52.9 (6.0)	$415 ($127)	$656 ($200)	$302 ($92)	$2,196 ($669)
12	48.7 (5.5)	155.6 (17.6)	34.1 (3.9)	$278 ($85)	$619 ($189)	$195 ($59)	$1,914 ($583)
13	37.0 (4.2)	133.7 (15.1)	24.0 (2.7)	$211 ($64)	$532 ($162)	$137 ($42)	$1,702 ($519)
40-year Return Period							
1	117.4 (13.3)	83.8 (9.5)	52.4 (5.9)	$670 ($204)	$333 ($102)	$299 ($91)	$2,171 ($662)
3	219.9 (24.9)	204.6 (23.2)	122.2 (13.8)	$1,256 ($383)	$814 ($248)	$698 ($213)	$3,681 ($1,122)
5	217.7 (24.7)	242.3 (27.5)	130.8 (14.8)	$1,243 ($379)	$964 ($294)	$747 ($228)	$3,867 ($1,179)
7	202.4 (22.9)	237.8 (26.9)	119.8 (13.6)	$1,156 ($352)	$946 ($288)	$684 ($208)	$3,699 ($1,127)
8	197.1 (22.3)	327.7 (37.1)	136.4 (15.5)	$1,126 ($343)	$1,303 ($397)	$779 ($237)	$4,122 ($1,256)
10	88.6 (10.0)	325.7 (36.9)	87.2 (9.9)	$506 ($154)	$1,295 ($395)	$498 ($152)	$3,121 ($951)
11	74.8 (8.5)	165.6 (18.8)	54.4 (6.2)	$427 ($130)	$659 ($201)	$311 ($95)	$2,219 ($676)
12	49.6 (5.6)	155.9 (17.7)	34.7 (3.9)	$283 ($86)	$620 ($189)	$198 ($60)	$1,923 ($586)
13	39.0 (4.4)	134.4 (15.2)	25.2 (2.9)	$223 ($68)	$534 ($163)	$144 ($44)	$1,723 ($525)

Armor stability

A number of trials were run with program LC_COST_REV to define armor damage throughout the life of the structure for historical wave and water level conditions. To isolate failure of the structure from stability, the results for a heavily armored crest were used so that there were no failures from overtopping. Table 50 summarizes breaches resulting from instability. Table 51 summarizes minor damage repairs resulting from instability. It is clear that there is no more than one failure or breach due to armor instability for all stations and all historical storms over 148 years if the structure cross sections are designed for return periods of 30 years or greater. All breach repairs are eliminated if the 100-year return period cross section is employed. There is no more than one minor damage repair due to armor instability for all stations if the structure cross sections are designed for return periods of 35 years or greater. Minor repairs resulting from armor instability are eliminated only if the 100-year return period design is used.

Note that for some stations and some return periods, the number of breach repairs actually increases with increasing design return period. Intuitively, one would expect the number of breaches to decrease or remain constant as the design return period increases. The reason for this behavior is as follows, using sta 1 as an example. For the 25-year return period design, two breaches occurred during the life cycle. Each of these breaches was due not only to a major storm attacking the structure, but also to one or more preceding storms that caused minor damage but not sufficient damage to initiate repairs. With the 30-year design, the first storm that breached the 25-year design was unable to breach the more robust structure but caused “minor” damage sufficient to trigger repairs. The second storm that breached the 25-year design then hit an intact structure and failed to breach it. With the 35-year design, storm #1 caused damage, but not as much as with the smaller stone in the 30-year design. The damage was not extensive enough to trigger a repair. Storm #2 then attacked a weakened structure and was able to breach it. This process is evident in the armor stability damage tables but not in the overtopping tables. That is because overtopping damage is not cumulative because there is no overtopping damage progression model, only limits. This interplay between cumulative minor and major damage and repairs reflects the life-cycle processes actually experienced by stone structures. It illustrates one of the key benefits of using ELS methods for design optimization.

Overtopping

For all stations, overtopping for both paved and unarmored crests was analyzed. Tables 52 and 53 list the number of repairs resulting from overtopping damage throughout the entire time history. For an unarmored crest, sta 2-7 can be grouped and sta 9-13 can be grouped. Stations 1 and 8 appear to be transition areas. For sta 2-7, the unarmored crest height required to avoid breach failures is 3.05 m (10 ft). Similarly, for sta 9-13, the unarmored crest height required to avoid breach failures is 2.44 m (8 ft). For a paved crest, Table 53 indicates that crest heights of 2.44 m (8 ft) or 2.74 m (9 ft) would be sufficient for sta 2-7, while a 2.13-m (7-ft) crest height would be sufficient for sta 9-13. However, the least-cost analysis showed the paved crest to have a slightly higher present-worth total cost than the unarmored crest.

Table 50
Number of Repairs of Breaches Due to Armor Instability as Function of Return Period for Historical Wave Conditions

Station	Return Period										
	5	10	15	20	25	30	35	40	45	50	100
1	11	2	2	1	2	0	1	1	1	0	0
2	12	5	3	1	1	0	0	1	0	0	0
3	14	4	2	1	0	0	0	0	0	0	0
4	10	2	1	0	0	0	0	0	0	0	0
5	7	2	0	0	0	0	0	0	0	0	0
6	7	2	1	0	0	0	0	0	0	0	0
7	5	1	1	0	0	0	0	0	0	0	0
8	9	4	1	0	0	0	0	0	0	0	0
9	5	2	3	0	1	1	1	1	0	0	0
10	6	0	0	0	0	0	0	0	0	0	0
11	5	2	1	1	0	0	0	0	0	0	0
12	7	0	1	0	0	0	0	0	0	0	0
13	7	3	3	1	2	1	1	0	1	1	0

Table 51
Number of Repairs of Minor Damage Due to Armor Instability as Function of Return Period for Historical Wave Conditions

Station	Return Period										
	5	10	15	20	25	30	35	40	45	50	100
1	4	6	2	2	0	2	0	0	0	1	0
2	3	3	3	3	1	2	1	0	1	0	0
3	4	3	1	2	3	1	1	1	0	0	0
4	5	3	2	2	1	1	0	0	0	0	0
5	6	1	3	2	1	1	1	1	1	0	0
6	3	2	1	1	1	1	1	1	1	1	0
7	7	5	1	1	1	1	1	1	1	1	0
8	2	2	2	2	2	1	0	0	0	0	0
9	10	4	0	3	1	0	0	0	0	0	0
10	3	3	1	1	1	0	0	0	0	0	0
11	5	4	2	2	2	2	1	0	0	0	0
12	5	3	0	1	1	0	0	0	0	0	0
13	8	3	1	3	0	1	0	1	0	0	0

Table 52
Number of Breaches Due to Overtopping for Unarmored Crest, Return Periods Greater than 35 Years, and Historical Wave Conditions

Station	Crest Height in m (ft)						
	1.8(6)	2.1(7)	2.4(8)	2.7(9)	3.0(10)	3.4(11)	3.7(12)
1	4	3	0	0	0	0	0
2	9	5	3	2	0	0	0
3	9	6	4	3	0	0	0
4	9	6	3	3	0	0	0
5	8	6	3	2	0	0	0
6	6	4	3	2	0	0	0
7	6	3	3	1	0	0	0
8	3	3	0	0	0	0	0
9	0	0	0	0	0	0	0
10	2	0	0	0	0	0	0
11	3	0	0	0	0	0	0
12	3	0	0	0	0	0	0
13	3	0	0	0	0	0	0

Table 53
Number of Breaches Due to Overtopping for Paved Crest, Return Periods Greater than 35 Years, and Historical Wave Conditions

Station	Crest Height in m (ft)						
	1.8(6)	2.1(7)	2.4(8)	2.7(9)	3.0(10)	3.4(11)	3.7(12)
1	3	0	0	0	0	0	0
2	3	3	1	0	0	0	0
3	6	3	0	0	0	0	0
4	6	3	0	0	0	0	0
5	4	3	0	0	0	0	0
6	3	3	0	0	0	0	0
7	3	3	0	0	0	0	0
8	3	0	0	0	0	0	0
9	0	0	0	0	0	0	0
10	1	0	0	0	0	0	0
11	1	0	0	0	0	0	0
12	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0

Compound slope runup

An analysis of the compound slope shown in Figure 60 of Chapter 6 was conducted to determine the maximum runup heights and armor requirements on the upland cells for historical wave conditions. Table 54 lists the results of 2 percent runup heights measured from the roadway up the upper slope for selected stations. Results for two roadway crest heights are listed. In general, the largest storms produce wave runup on the upper slope of the upland cells. At this time, sta 7-13 are being considered for the upland cell configuration. The maximum runup height is 2.5 m (8.2 ft) for the low crest height at sta 7. Station 7 is the only upland location with open exposure to storm waves. Other stations showed considerably less runup on the upper slope. Runup for the upper slope at sta 8 is roughly half the maximum 2 percent runup at sta 7. Stations 10-13 show 1.2 m (3.9 ft) or less runup for the low crest height of 1.83 m (6 ft) and no runup for the highest crest height of 3.05 m (10 ft). Where armoring is required, the material would not need to exceed a height of 1.83 m (6 ft) on the upland slope except at sta 7, where the upper slope would require armoring to an elevation of 3.05 m (10 ft) above the roadway.

Assuming runup to be equivalent to 1.5-3.0 times the wave height, a stable stone size can be calculated. Stable stone weights computed using $H_s = R_u/2$ are listed in Table 54. For example, an armor stone weight of 1,892 N (425 lb) would be stable on the slope of 1V:3H at sta 7 with the low crest. The required stone size drops dramatically as one progresses toward sta 10. From sta 10 around the eastern side of the island, the upland armor stone sizes are negligible.

Table 54
Maximum Runup Heights on Upland Cell Slopes

Station	Crest Height (of Roadway), m (ft)	Upland Runup R_{uc}, m (ft)	Stable Armor Weight, N (lb)
1	1.83 (6)	1.09 (3.6)	159 (36)
	3.05 (10)	0.37 (1.2)	6 (1)
3	1.83 (6)	2.50 (8.2)	1915 (430)
	3.05 (10)	1.50 (4.9)	414 (93)
5	1.83 (6)	2.88 (9.4)	2927 (658)
	3.05 (10)	1.80 (5.9)	715 (161)
7	1.83 (6)	2.49 (8.2)	1892 (425)
	3.05 (10)	1.53 (5.0)	439 (99)
8	1.83 (6)	1.43 (4.7)	358 (81)
	3.05 (10)	0.69 (2.3)	40 (9)
10	1.83 (6)	0.29 (1.0)	3 (1)
	3.05 (10)	0.00 (0.0)	0 (0)
11	1.83 (6)	0.32 (1.0)	4 (1)
	3.05 (10)	0.00 (0.0)	0 (0)
12	1.83 (6)	0.18 (0.6)	1 (0)
	3.05 (10)	0.00 (0.0)	0 (0)
13	1.83 (6)	0.37 (1.2)	6 (1)
	3.05 (10)	0.00 (0.0)	0 (0)

Summary of structural response to historical waves and water levels

Given this preliminary analysis using only historical wave and water level conditions, the optimal cross sections corresponding to Figure 58 are as follows:

a. Stations 2-7: Unarmored crest height = 3.05 m (10 ft).

b. Stations 9-13: Unarmored crest height = 2.44 m (8 ft).

c. Stations 1 and 8 would be transition areas for crest height.

d. Structure slope: 1V:3H.

e. Toe crest height: +0.305 m (1 ft) mllw.

f. Seaward toe slope: 1V:2H.

g. Leeward toe slope: 1V:1.5H.

h. Toe crest width: $4D_{toe.}$

The following lower-slope armor sizes are optimal for this preliminary study:

a. Stations 2-9:

(1) W_{a50} = 11,121 N (2,500 lb).

(2) W_{u50} = 1,112 N (250 lb).

(3) $W_{a,toe}$ = 15,569 N (3,500 lb).

b. Stations 1 and 10-12:

(1) W_{a50} = 2,224 N (500 lb).

(2) W_{u50} = 222 N (50 lb).

(3) $W_{a,toe}$ = 8,896 N (2,000 lb).

c. Station 13 and remainder of east side:

(1) W_{a50} = 222 N (50 lb).

(2) W_{u50} = 22 N (5 lb).

(3) $W_{a,toe}$ = 4,448 N (1,000 lb).

Final Analysis Using ELS Simulations

The previous sections focus the final analysis. In this section, the Empirical Life-cycle Simulation, or ELS, technique is used. For this analysis, 50 simulations of a 50-year wave and water level climate were generated for each design analysis station. Each wave time series was run through LC_COST_REV for four crest heights, all relative to mllw: 1.83 m (6.0 ft), 2.13 m (7.0 ft), 2.59 m (8.5 ft), and 3.05 m (10.0 ft). The fixed parameters for this portion of the study are listed in Table 55.

Table 55 Fixed Parameter Values for Final ELS Analysis		
Parameter	**Variable**	**Value**
Permeability	P	0.1
Porosity	Por	0.38
Stone specific gravity	S_r	2.578
Stone density	ρ_r	2.644 tonne/cu m (165 pcf)
Minor repair limit	S_M	8
Breach repair limit	S_B	18
Minor repair time limit	-	180 days
Breach repair time limit	–	120 days
Roughness parameter	γ_b	0.55
Crest width	B	7.62 m (25 ft)
Lower structure slope	α	1V:3H
Upper structure slope	α_2	1V:3H
Toe berm height	d_B	+0.305 m (1 ft) mllw
Toe berm seaward slope	cot φ	2
Toe berm leeward slope	cot β	1.5
Toe berm crest width	-	$4D_{toe}$
Toe armor thickness	-	$2D_{toe}$
Overtopping limit		0.05 cu m/sec/m
Allowable main armor damage	S	1.0
Allowable toe damage	N_{od}	1.0
Number of waves for zero damage	N_z	7000
Inflation or escalation rate	i	0.03
Interest rate	R	0.05375
Economic life	N	50 years
Armor material unit cost	-	$56/tonne ($50.4/ton)
Filter material unit cost	-	$39/tonne ($35.1/ton)
Bedding material unit cost	-	$44/tonne ($39.6/ton)
Quarry-run material unit cost	-	$44/tonne ($39.6/ton)
Geotechnical material unit cost	-	$4.78/sq m ($0.44/sq ft)
Lag before initial construction	Lag	0 years
Fixed first cost	FFC/L_s	$500/m ($152/ft)
Fixed repair cost	RFC/L_r	$2500/m ($762/ft)
Ratio of repair length to section length	L_r/L_s	0.3

All empirical simulations of wave and water level life cycles were compared to the historical to assure that they were statistically similar. In particular, time series, histograms, and cumulative distributions were plotted for H_s, T_p, water level, and wave direction for each simulation. In addition, the upper tails of the histograms and empirical cumulative distributions were examined to assure that the extreme values were being reproduced. In all cases, the distributions of historical time series were generally well reproduced in the simulations. However, only a small number of simulations reproduced the tails of the distributions well. Only the simulations that matched the entire historical distributions were used.

Figures 104-143 show simulation results summarized for key stations. The results shown are for the average of the simulation distributions that best matched the historical distributions. For each station the following are plotted as a function of return period:

a. Total present-worth cost for unarmored crest.

b. Total number of repairs for unarmored crest.

c. Total present-worth cost for armored crest.

d. Total number of repairs for armored crest.

e. Number of breach repairs due to armor damage.

f. Number of minor repairs due to armor damage.

For the more sheltered stations, there may not be damages for the armored section (sta 10-13). In this case, the plots are not shown.

Figure 104 shows the total present-worth costs for a sta 1 unarmored crest section at four crest heights. The two higher crests have lower costs because of fewer repairs. The minimum-cost return period design is at 20 years for this section. However, there is little cost penalty for the higher return period designs. Figure 105 summarizes the total number of repairs for the unarmored crest over the 50-year life cycle for the various return period designs with varying crest heights. The lower crests all have significant repairs. The two higher crests have little risk of requiring repairs during the design life for design return periods of 20 years or greater. Figures 106-109 show the armored crest performance for sta 1. The least-cost alternative has a similar cost to the unarmored crest alternative. However, the armored crest is not vulnerable to breach failure due to overtopping. Figures 107-109 show that the armored low crest alternative would not fail for return period designs of 20 years or greater. Stations 1-8 shown in Figures 104-133 all have similar responses. Generally, these sections have the following properties:

a. The least-cost unarmored and armored crest sections cost about the same.

b. The cost curves are fairly flat for higher return period designs. There is little cost penalty to going with a more conservative armor stone size.

c. Higher crests of 3.05 m (10 ft) are required to avoid breaching of unarmored crests. Armored crests can be built lower and have a much lower probability of breaching.

Stations 10-13 shown in Figures 134-143 are more sheltered. The cost curves tend to be monotonically increasing or have minima at very low return periods. The least-cost alternative for these stations tends to be the unarmored crest at elevation 2.13 m (7 ft). There are no repairs required for return periods greater than about 15 years for this alternative.

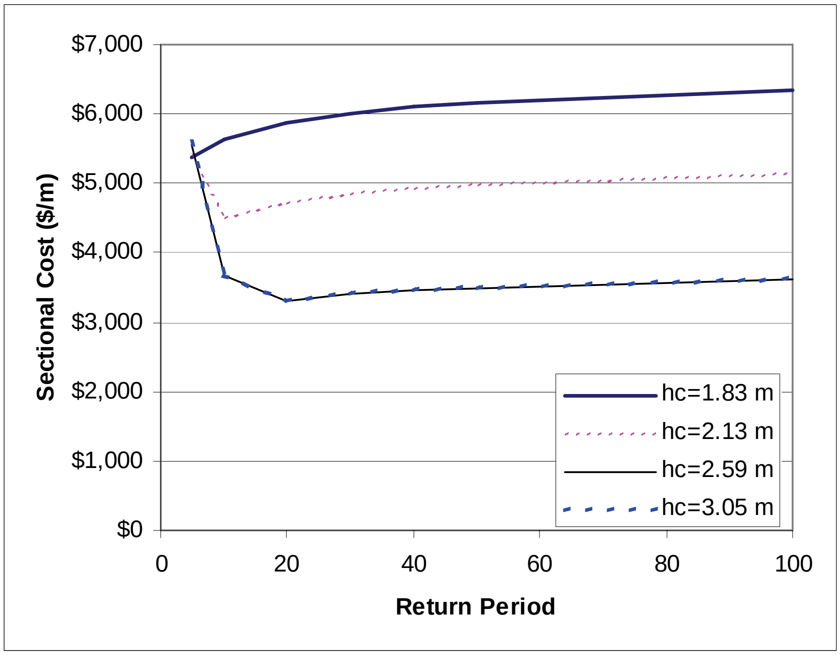


Figure 104. Total cross-sectional cost as function of return period at sta 1 for several crest heights. Crests were unarmored. Conversion: Cost/ft = Cost/m*0.3048 m/ft, h_c = 1.83 m (6.0 ft), h_c = 2.13 m (7.0 ft), h_c = 2.59 m (8.5 ft), h_c = 3.05 m (10.0 ft)

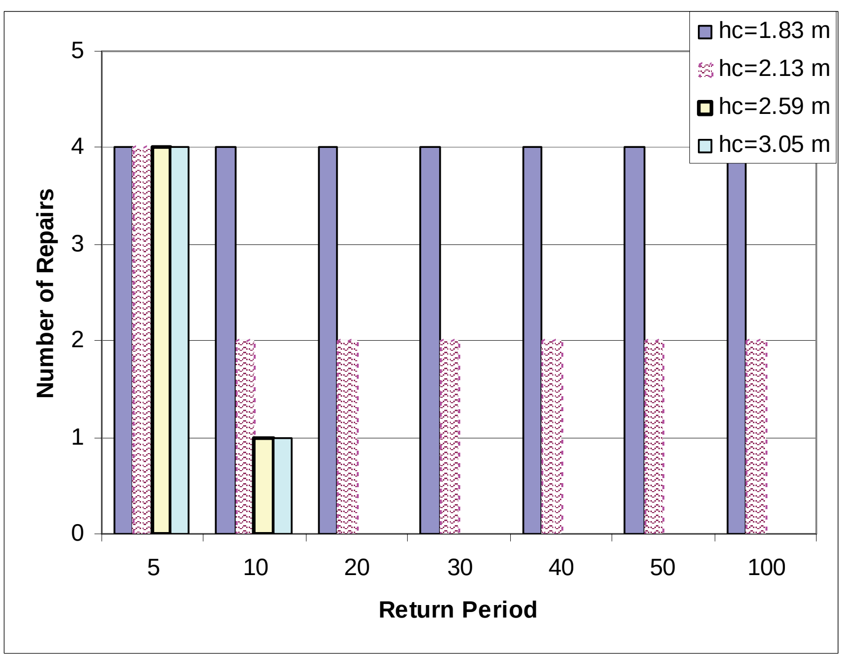


Figure 105. Total number of repairs as function of return period at sta 1 for several crest heights. Crests were unarmored

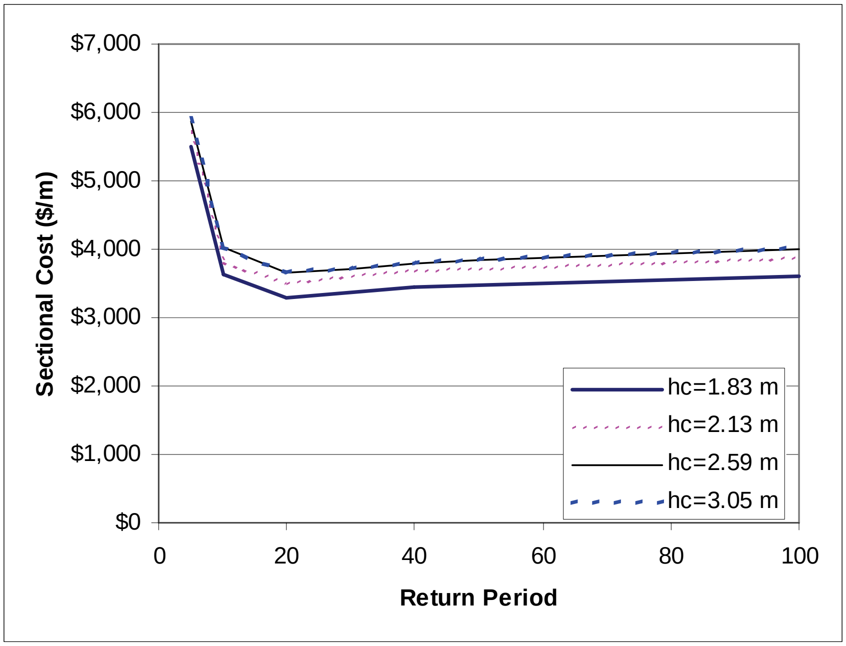


Figure 106. Total cross-sectional cost as function of return period at sta 1 for armored crests

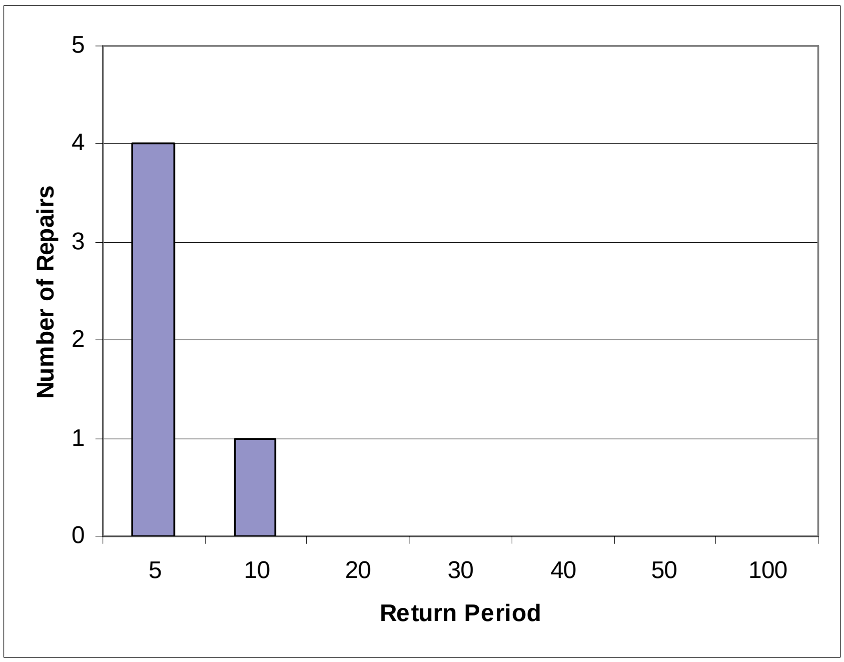


Figure 107. Total number of repairs as function of return period at sta 1 for armored crests

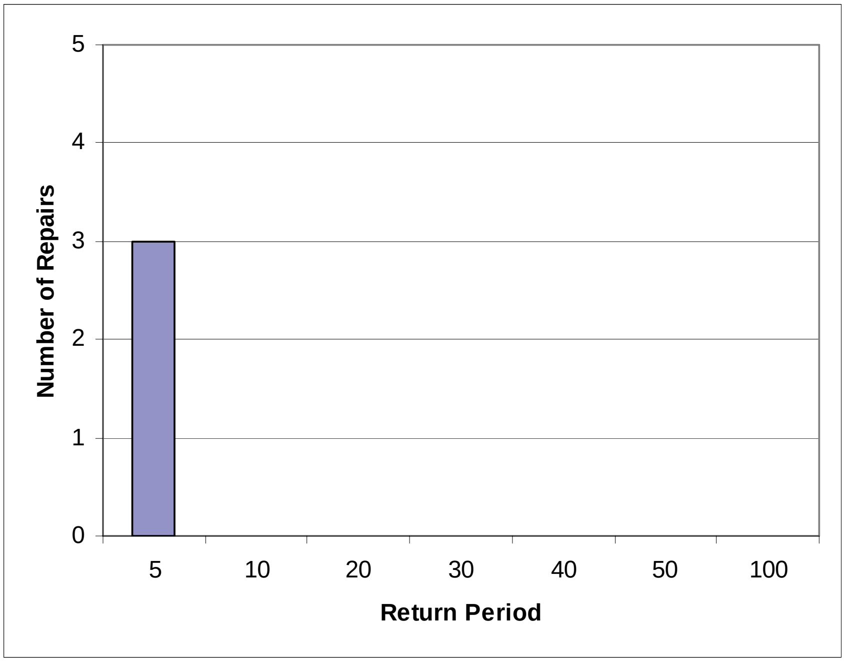


Figure 108. Number of breach repairs due to armor damage as function of return period at sta 1 for armored crests

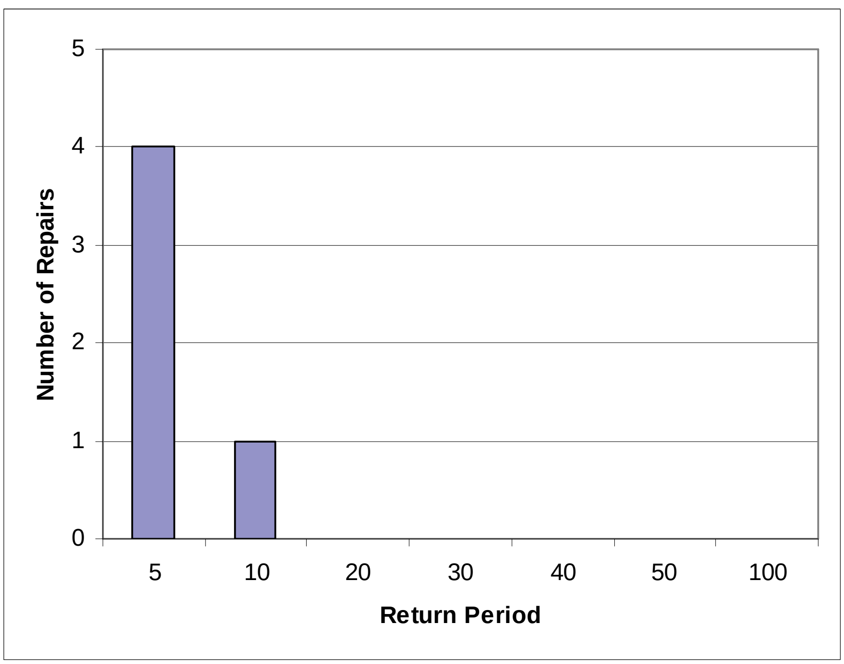


Figure 109. Number of minor repairs due to armor damage as function of return period at sta 1 for armored crests

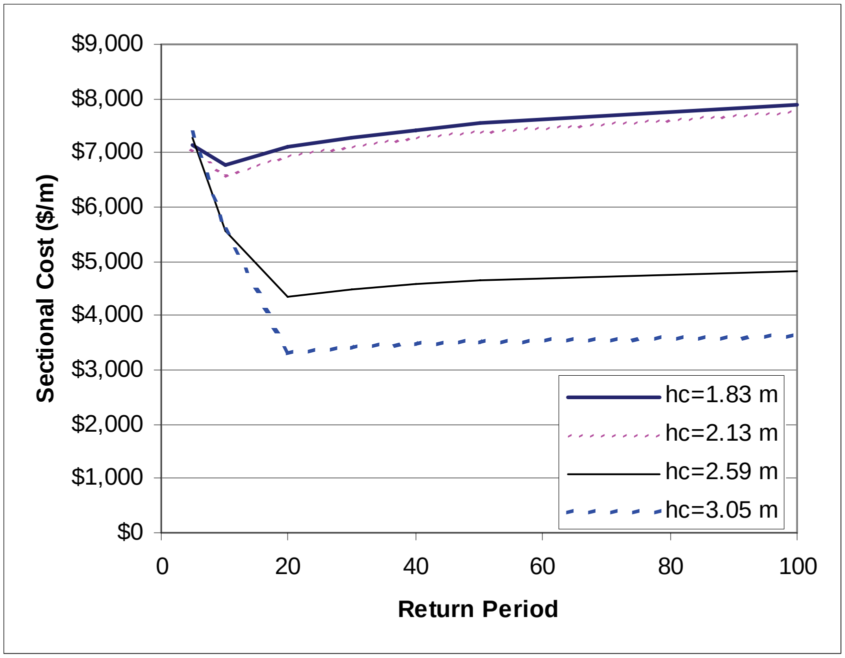


Figure 110. Total cross-sectional cost as function of return period at sta 3 for several crest heights. Crests were unarmored

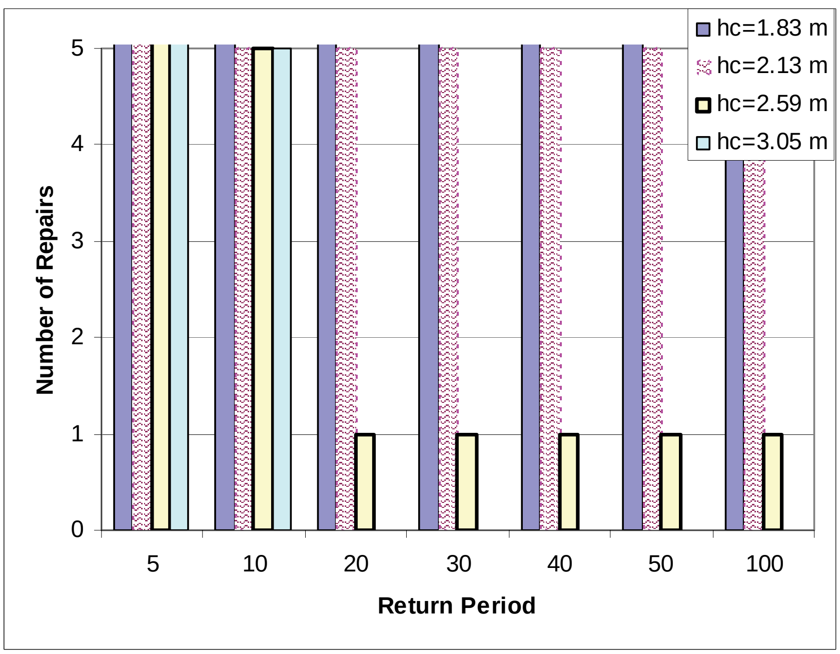


Figure 111. Total number of repairs as function of return period at sta 3 for several crest heights. Crests were unarmored

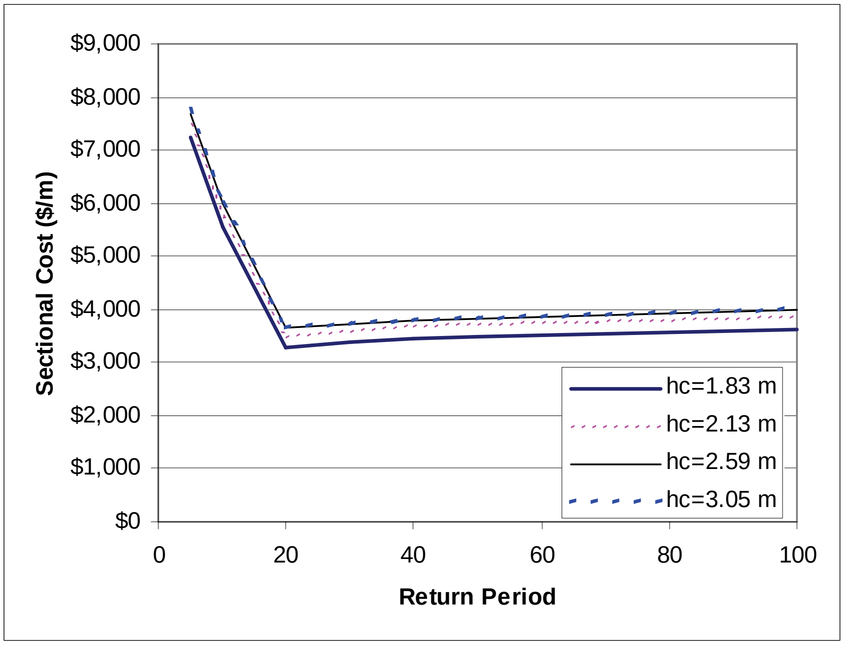


Figure 112. Total cross-sectional cost as function of return period at sta 3 for armored crests

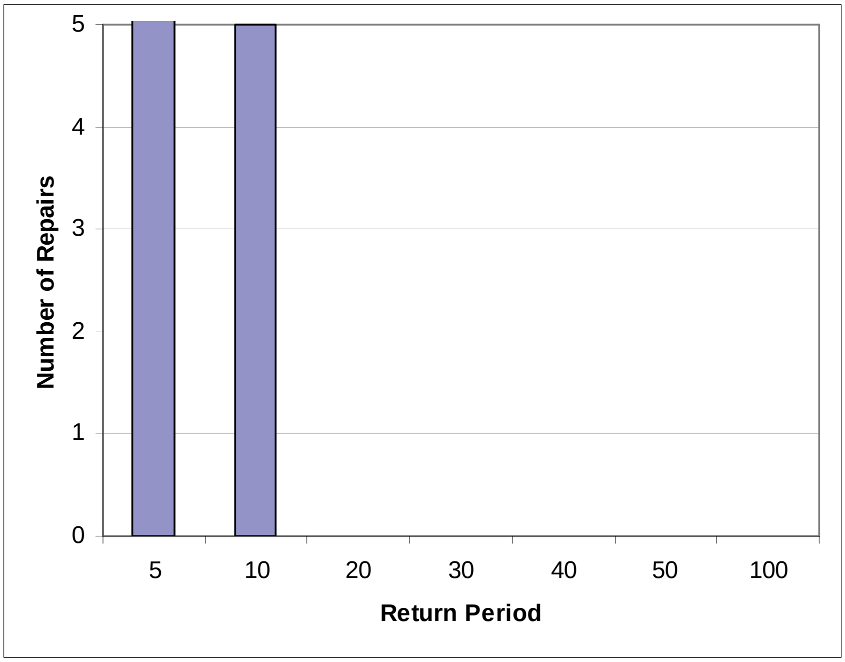


Figure 113. Total number of repairs due to armor damage as function of return period at sta 3 for armored crests

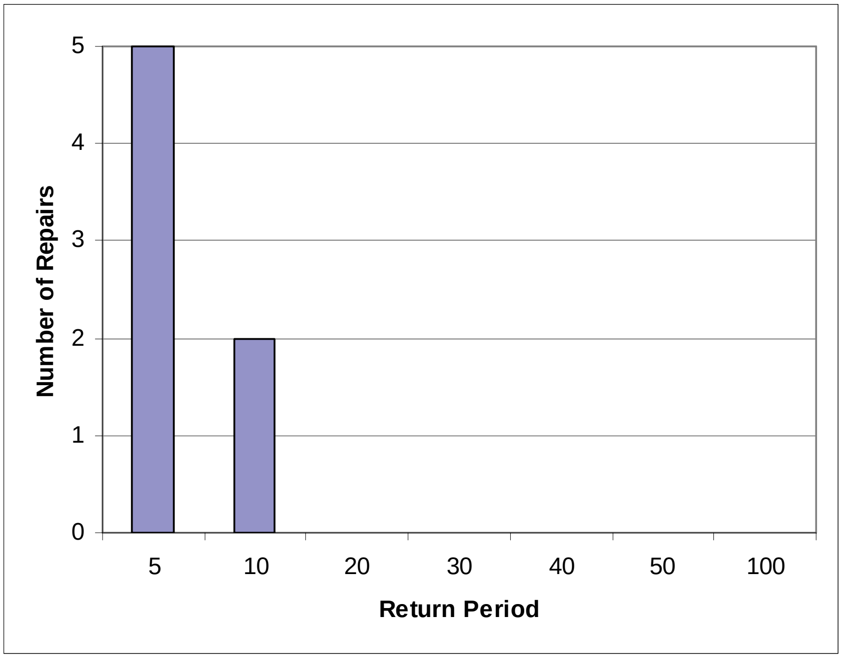


Figure 114. Number of breach repairs due to armor damage as function of return period at sta 3 for armored crests

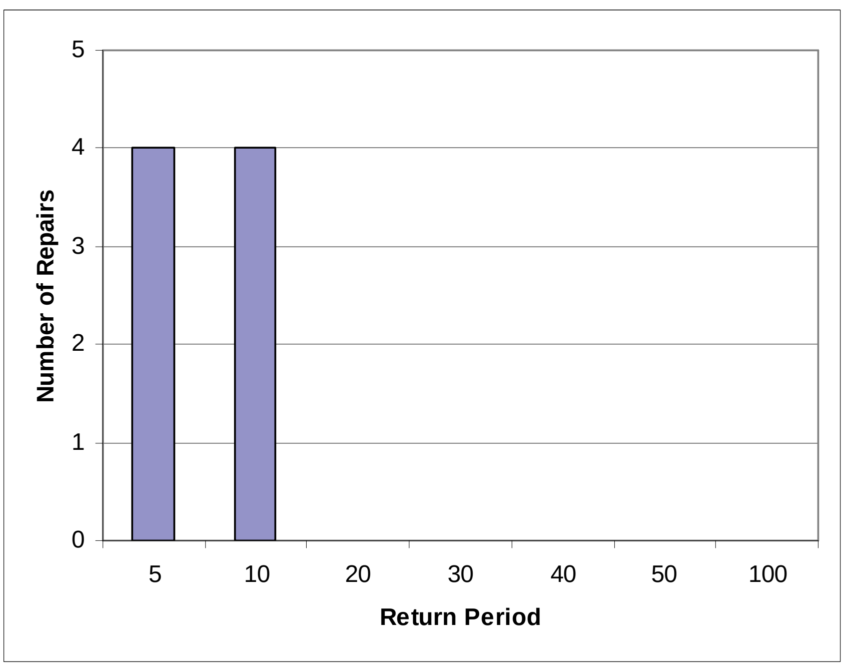


Figure 115. Number of minor repairs due to armor damage as function of return period at sta 3 for armored crests

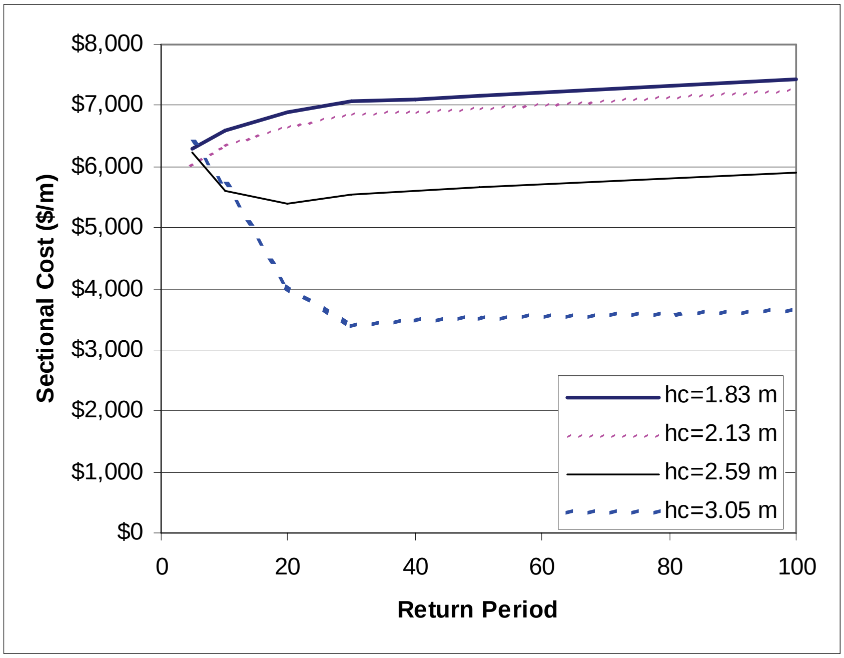


Figure 116. Total cross-sectional cost as function of return period at sta 5 for several crest heights. Crests were unarmored

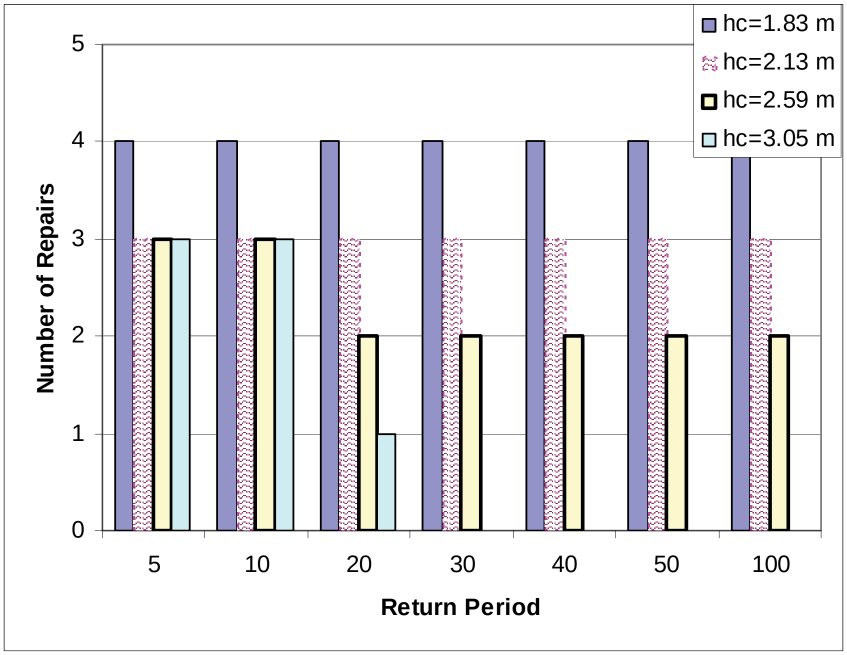


Figure 117. Total number of repairs as function of return period at sta 5 for several crest heights. Crests were unarmored

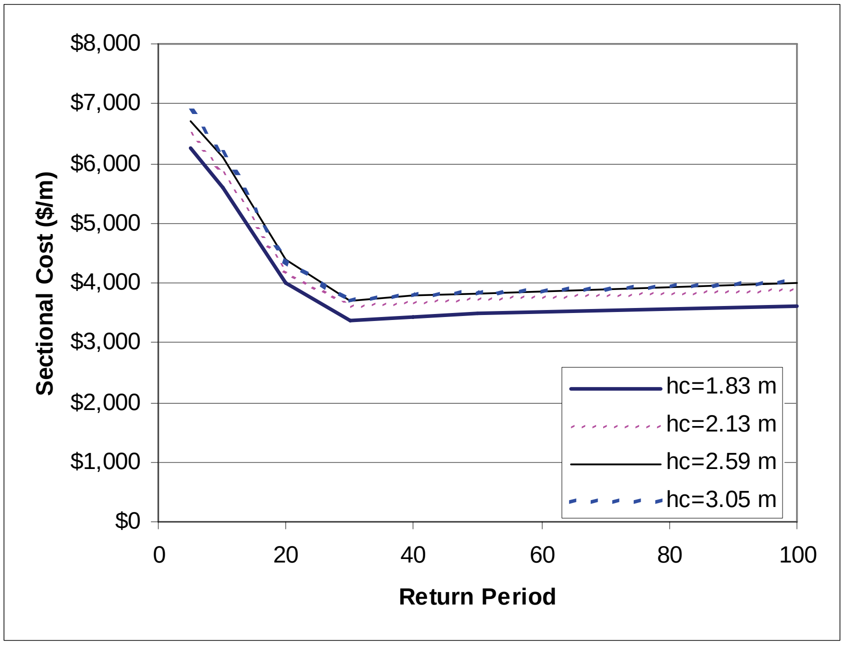


Figure 118. Total cross-sectional cost as function of return period at sta 5 for armored crests

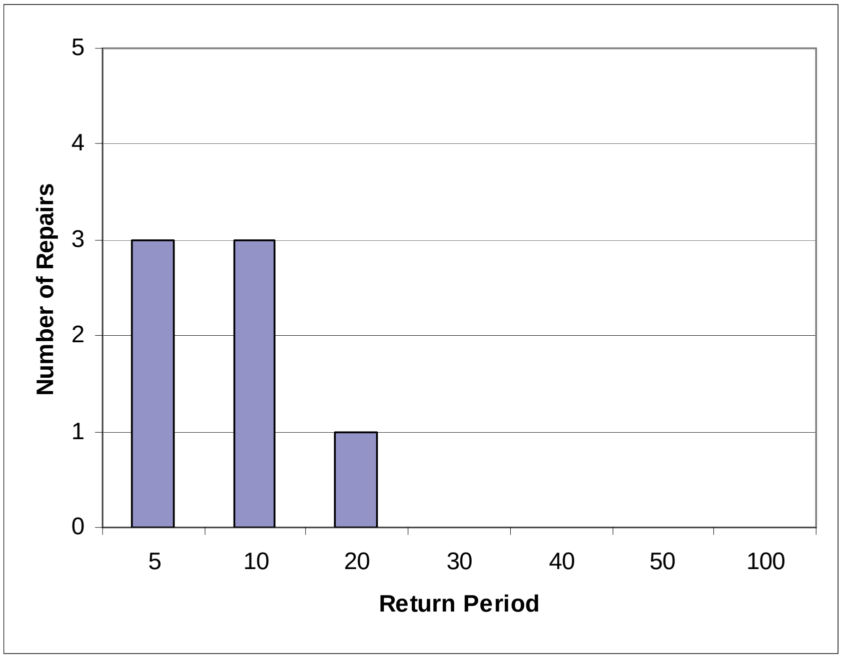


Figure 119. Total number of repairs due to armor damage as function of return period at sta 5 for armored crests

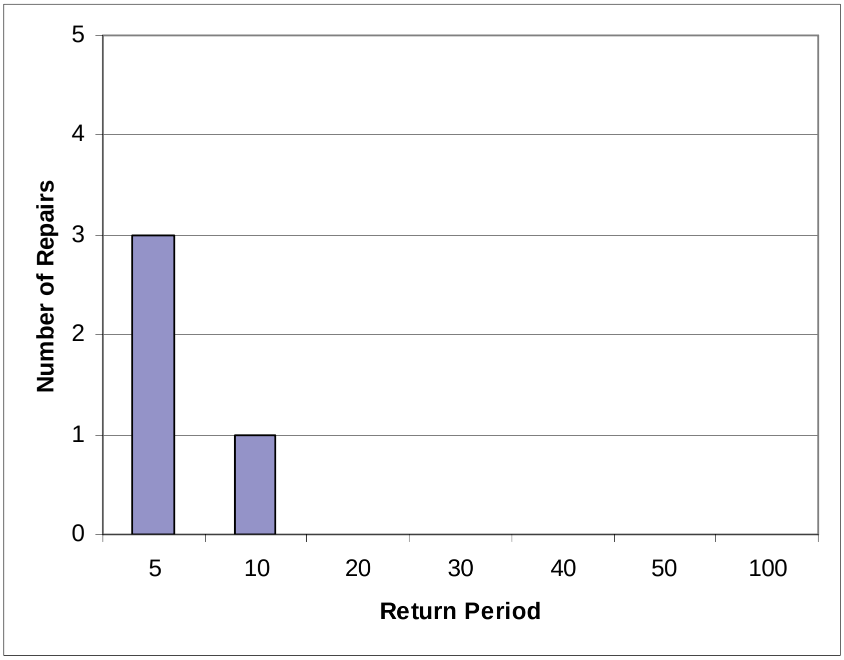


Figure 120. Number of breach repairs due to armor damage as function of return period at sta 5 for armored crests

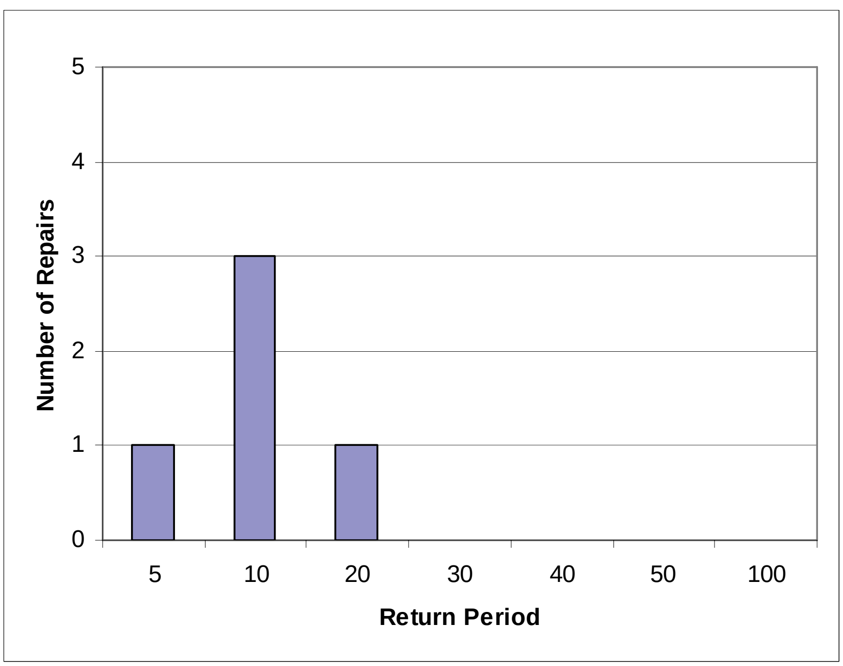


Figure 121. Number of minor repairs due to armor damage as function of return period at sta 5 for armored crests

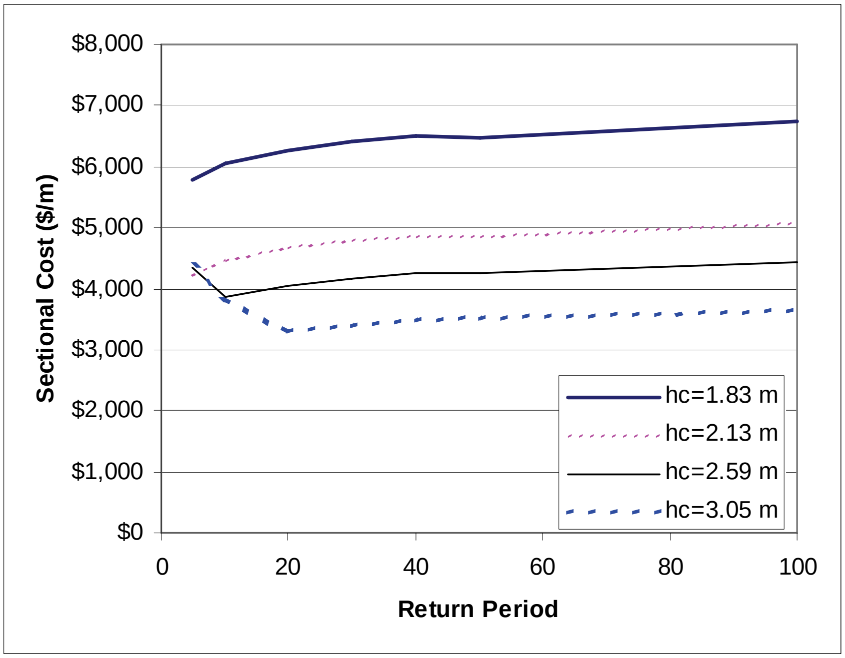


Figure 122. Total cross-sectional cost as function of return period at sta 7 for several crest heights. Crests were unarmored

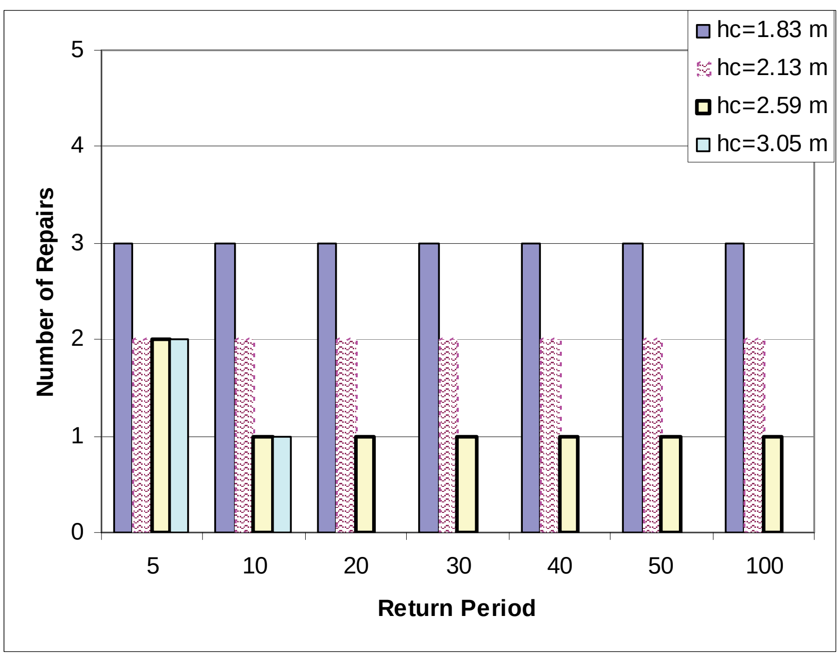


Figure 123. Total number of repairs as function of return period at sta 7 for several crest heights. Crests were unarmored

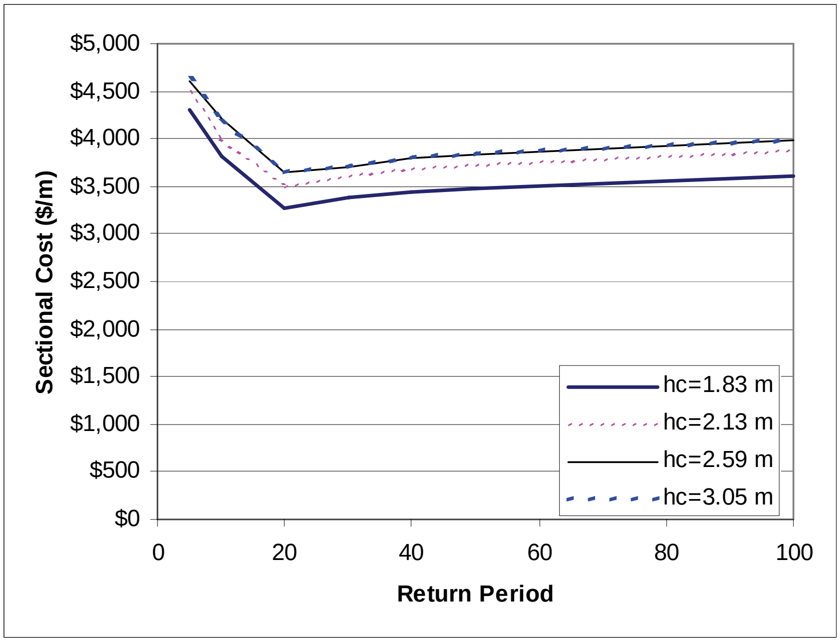


Figure 124. Total cross-sectional cost as function of return period at sta 7 for armored crests

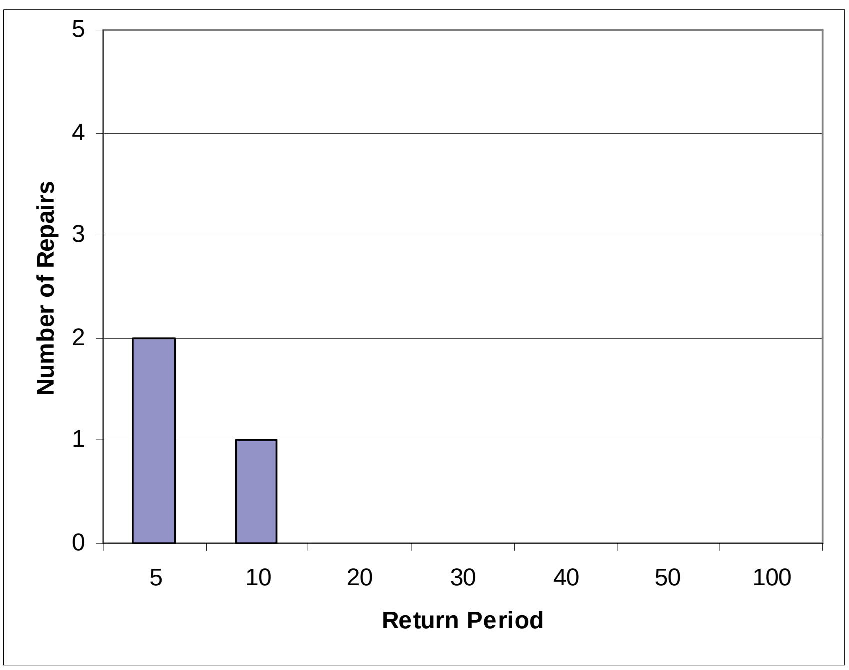


Figure 125. Total number of repairs due to armor damage as function of return period at sta 7 for armored crests

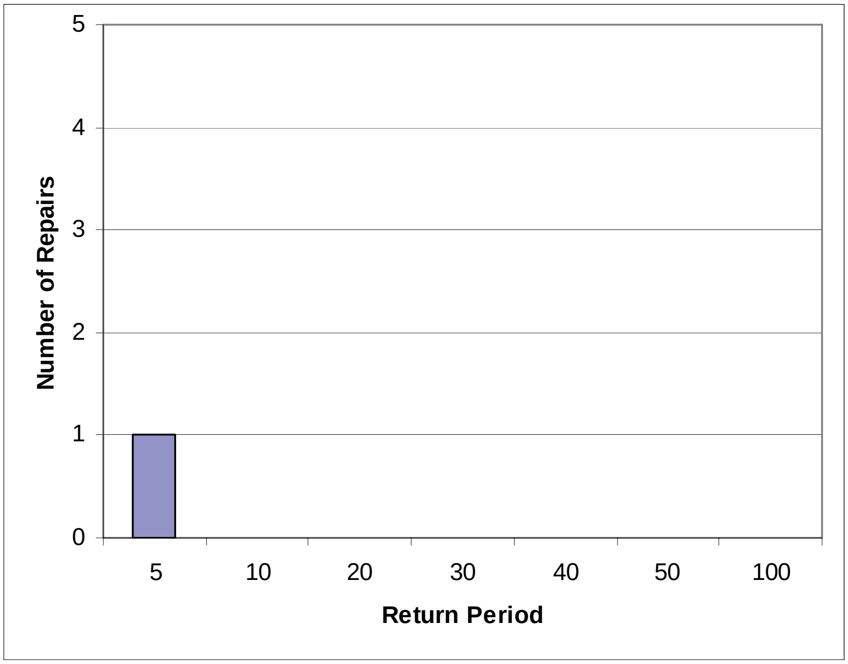


Figure 126. Number of breach repairs due to armor damage as function of return period at sta 7 for armored crests

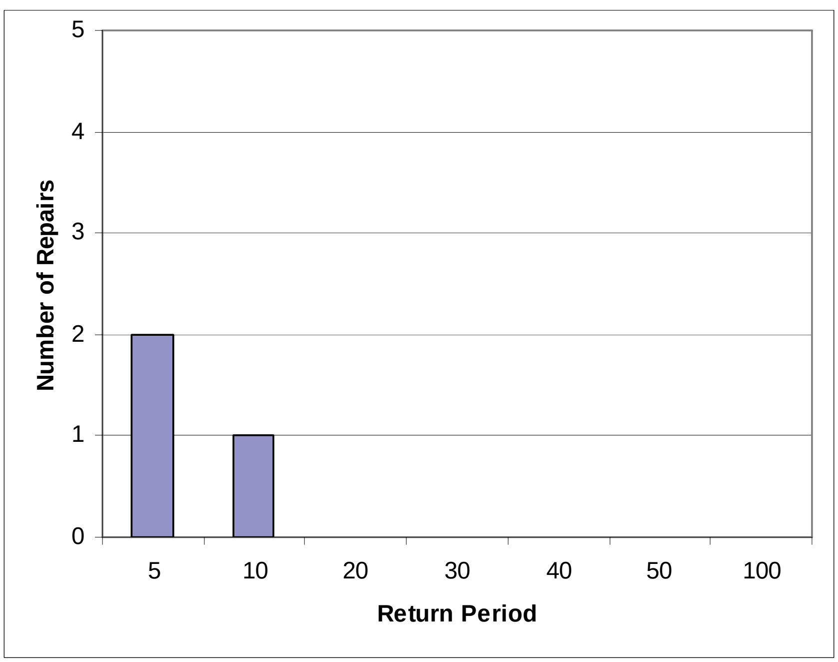


Figure 127. Number of minor repairs due to armor damage as function of return period at sta 7 for armored crests

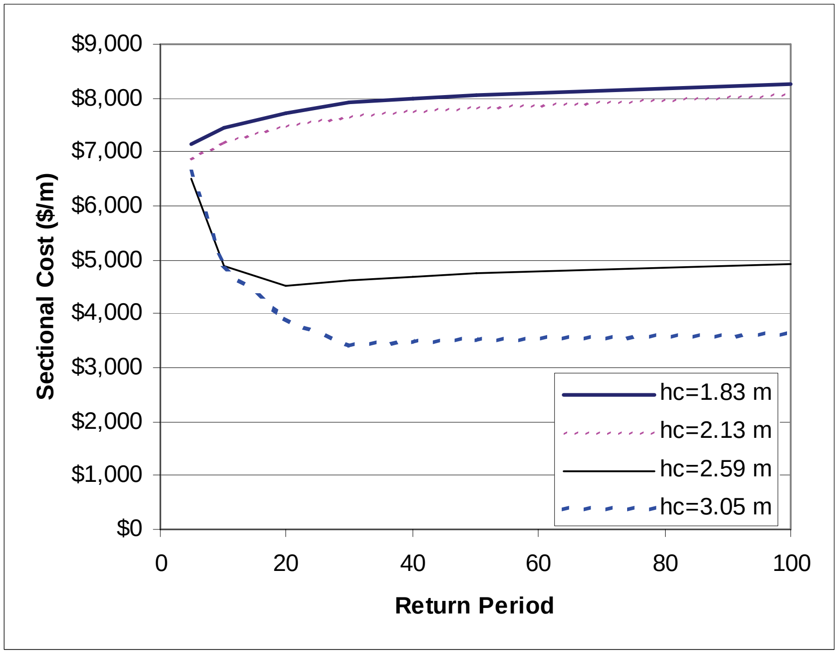


Figure 128. Total cross-sectional cost as function of return period at sta 8 for several crest heights. Crests were unarmored

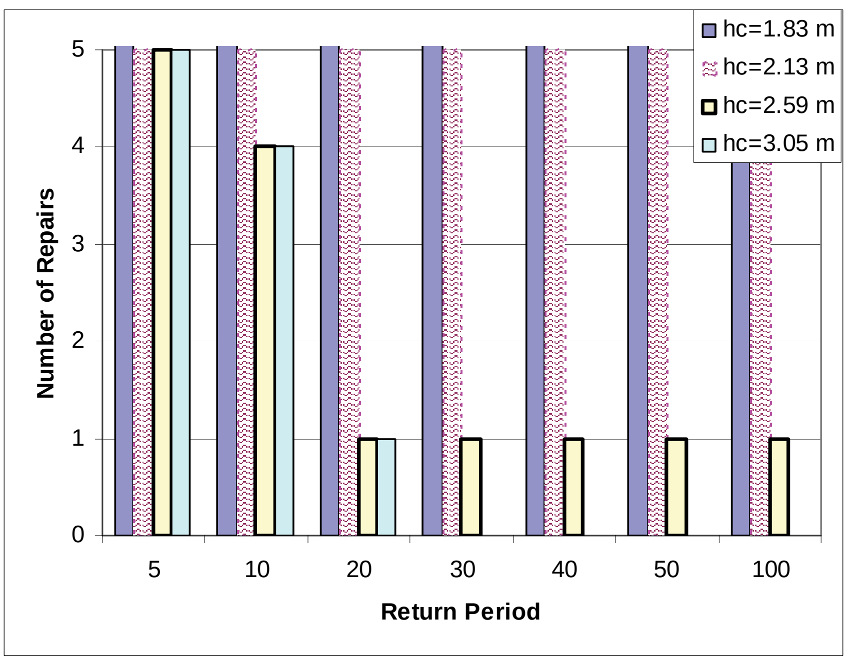


Figure 129. Total number of repairs as function of return period at sta 8 for several crest heights. Crests were unarmored

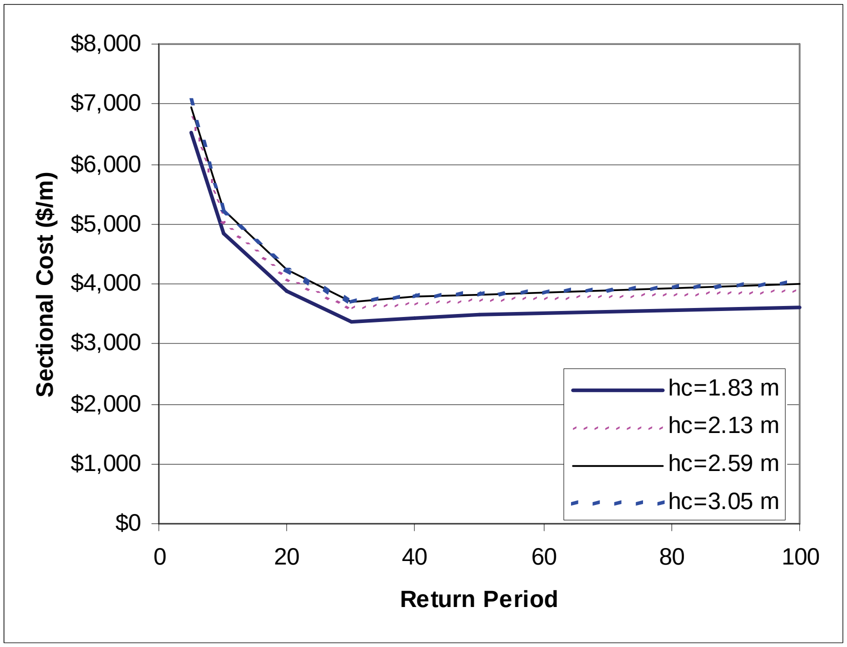


Figure 130. Total cross-sectional cost as function of return period at sta 8 for armored crests

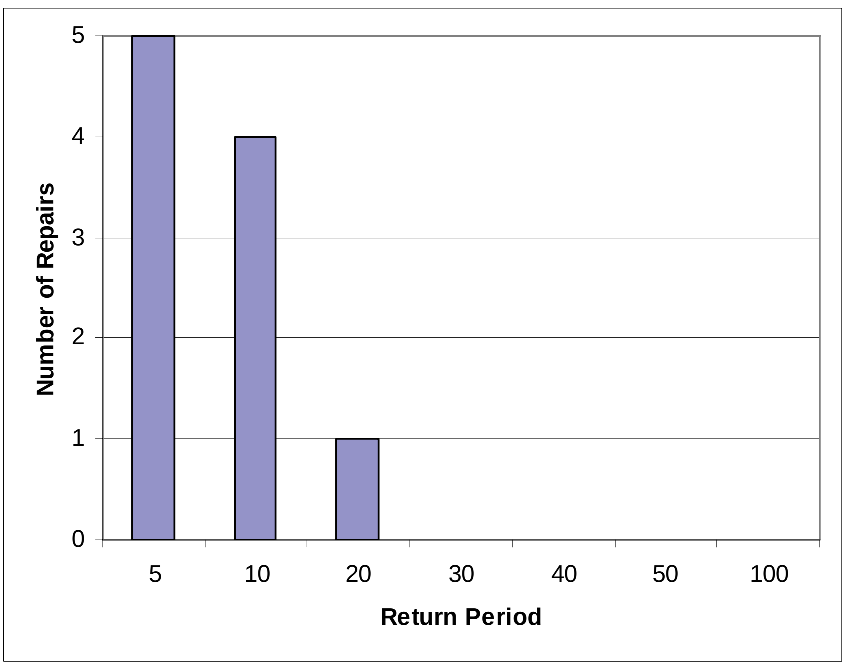


Figure 131. Total number of repairs due to armor damage as function of return period at sta 8 for armored crests

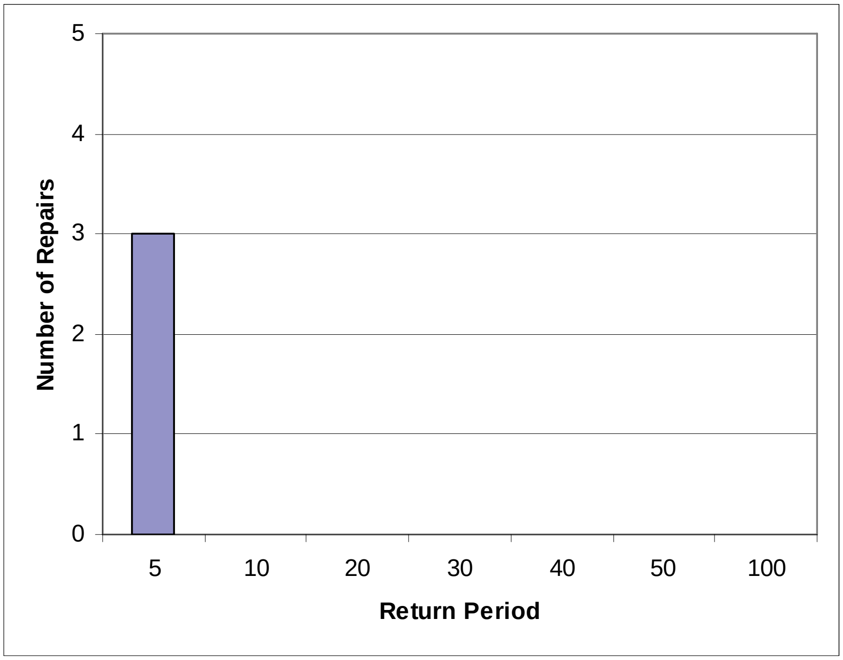


Figure 132. Number of breach repairs due to armor damage as function of return period at sta 8 for armored crests

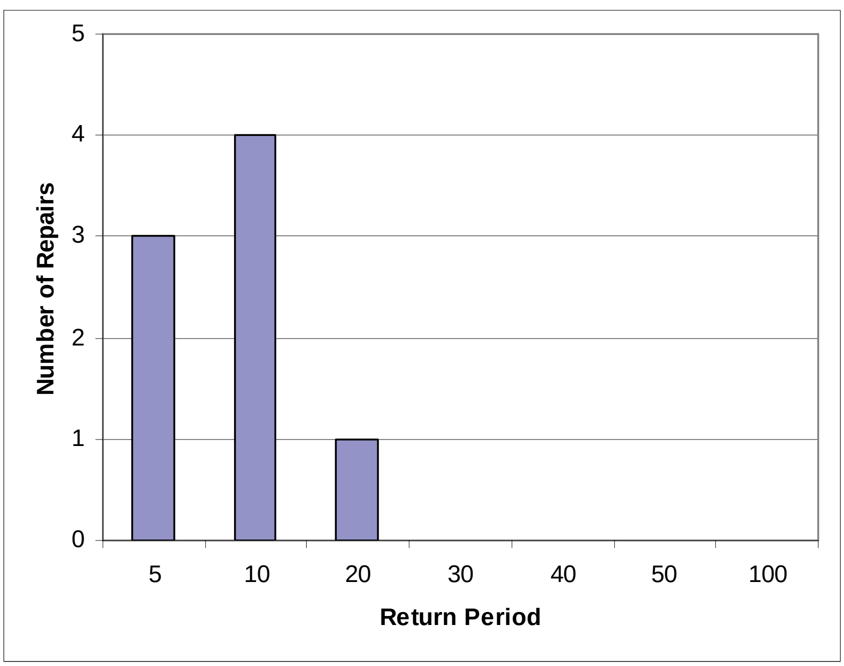


Figure 133. Number of minor repairs due to armor damage as function of return period at sta 8 for armored crests

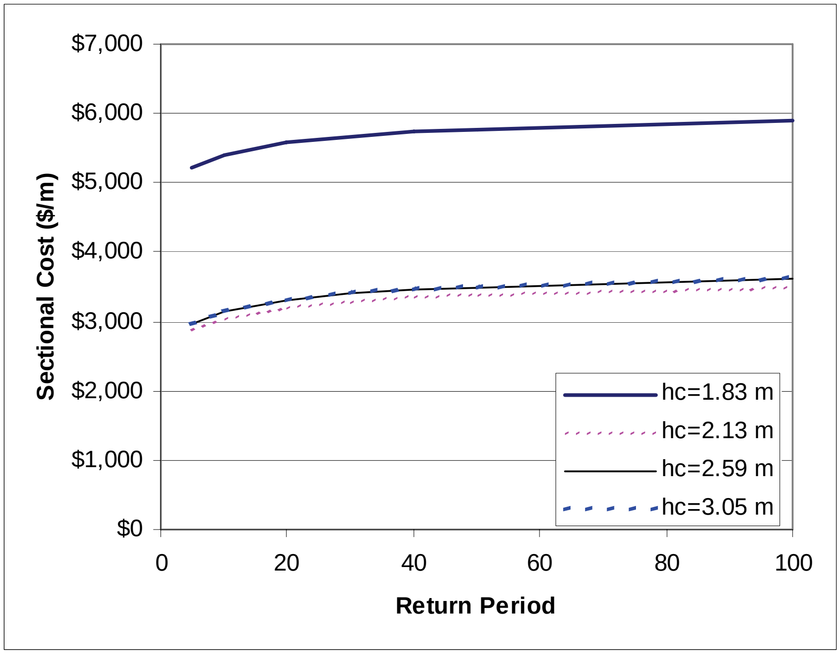


Figure 134. Total cross-sectional cost as function of return period at sta 10 for several crest heights. Crests were unarmored

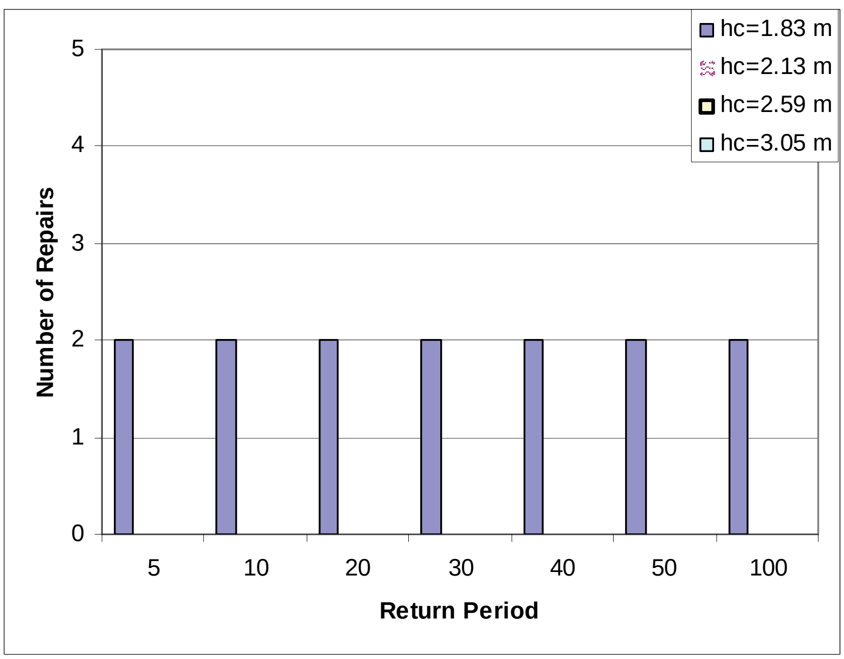


Figure 135. Total number of repairs as function of return period at sta 10 for several crest heights. Crests were unarmored

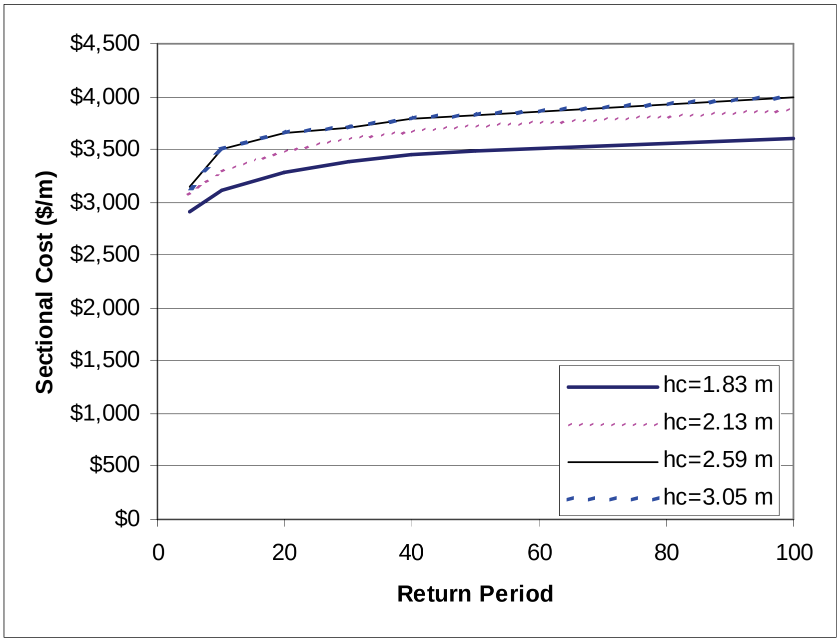


Figure 136. Total cross-sectional cost as function of return period at sta 10 for armored crests

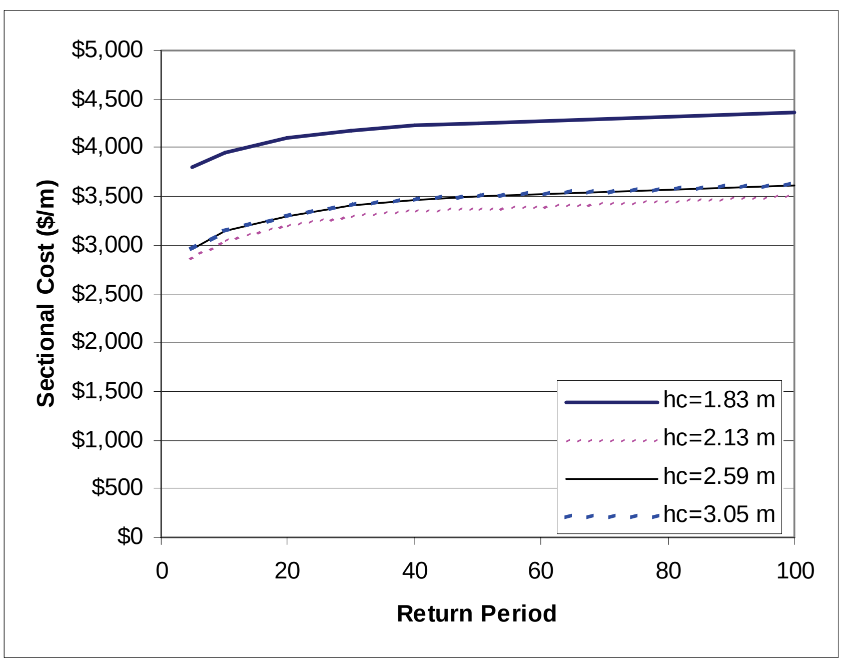


Figure 137. Total cross-sectional cost as function of return period at sta 12 for several crest heights. Crests were unarmored

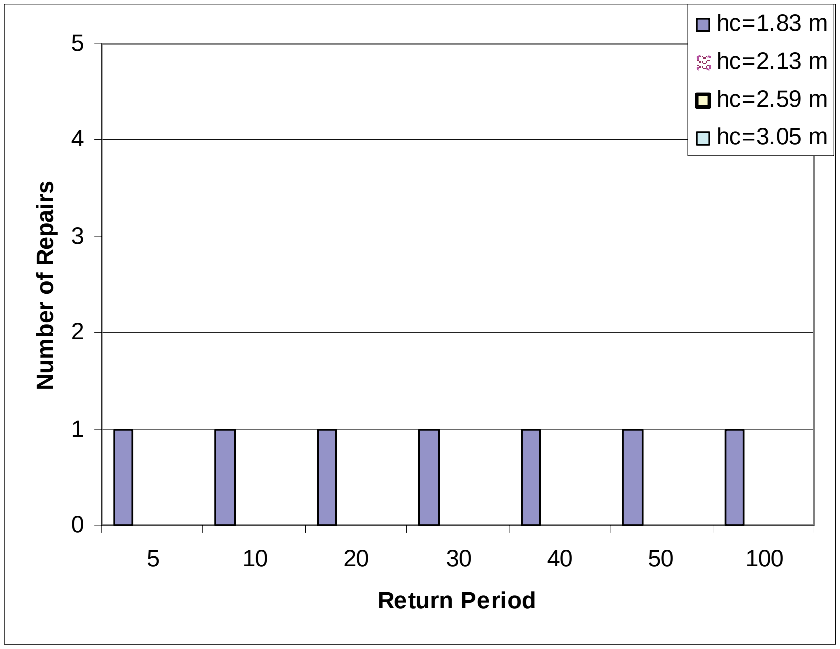


Figure 138. Total number of repairs as function of return period at sta 12 for several crest heights. Crests were unarmored

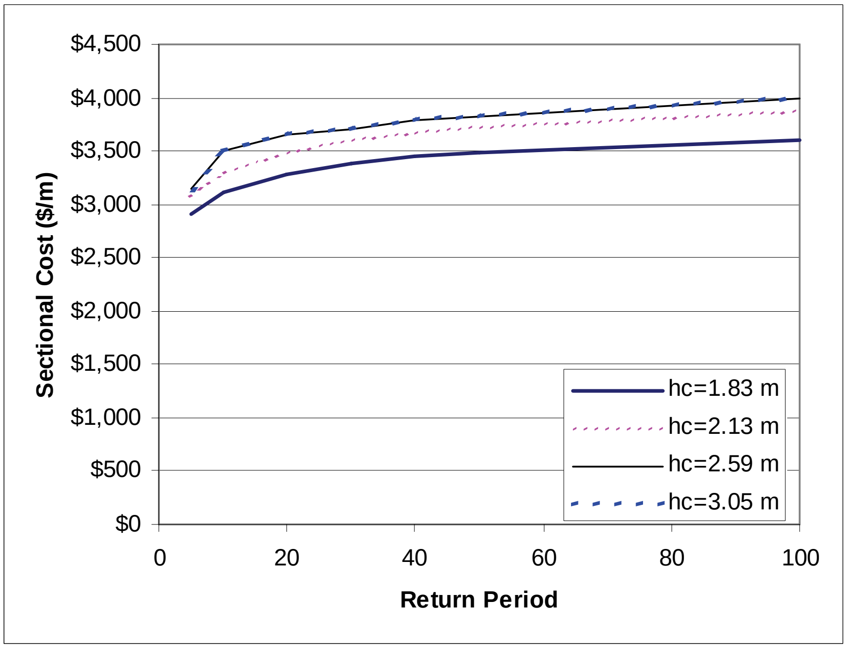


Figure 139. Total cross-sectional cost as function of return period at sta 12 for armored crests

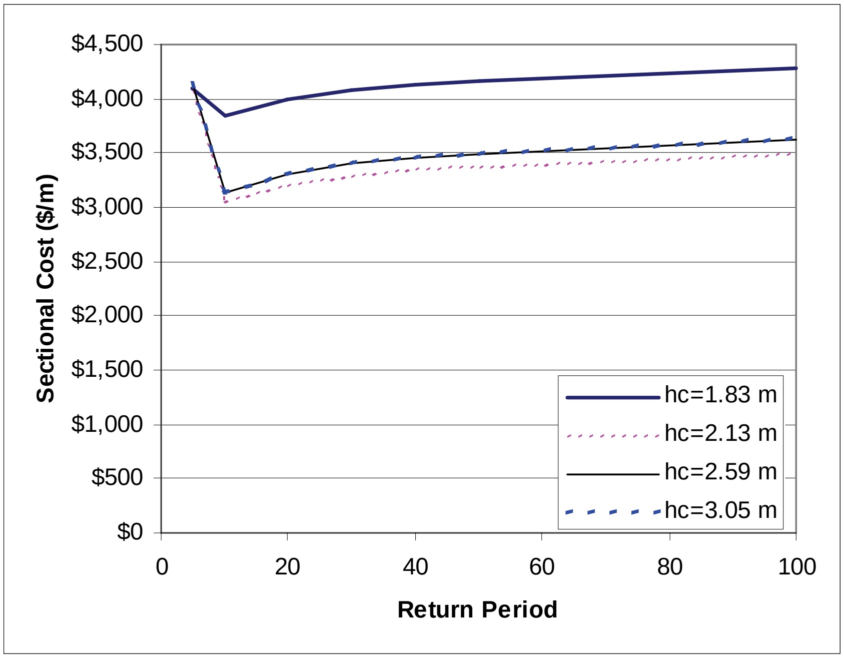


Figure 140. Total cross-sectional cost as function of return period at sta 13 for several crest heights. Crests were unarmored

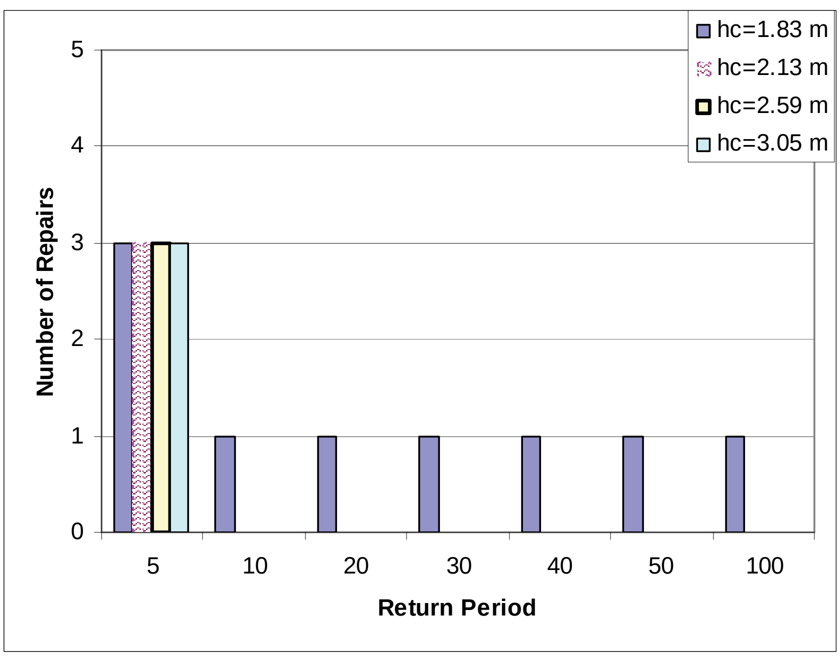


Figure 141. Total number of repairs as function of return period at sta 13 for several crest heights. Crests were unarmored

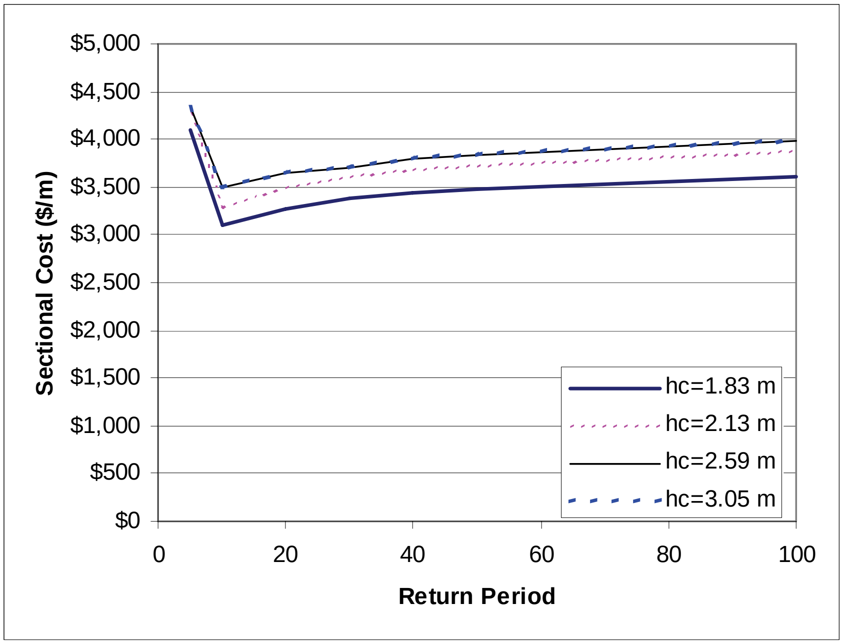


Figure 142. Total cross-sectional cost as function of return period at sta 13 for armored crests

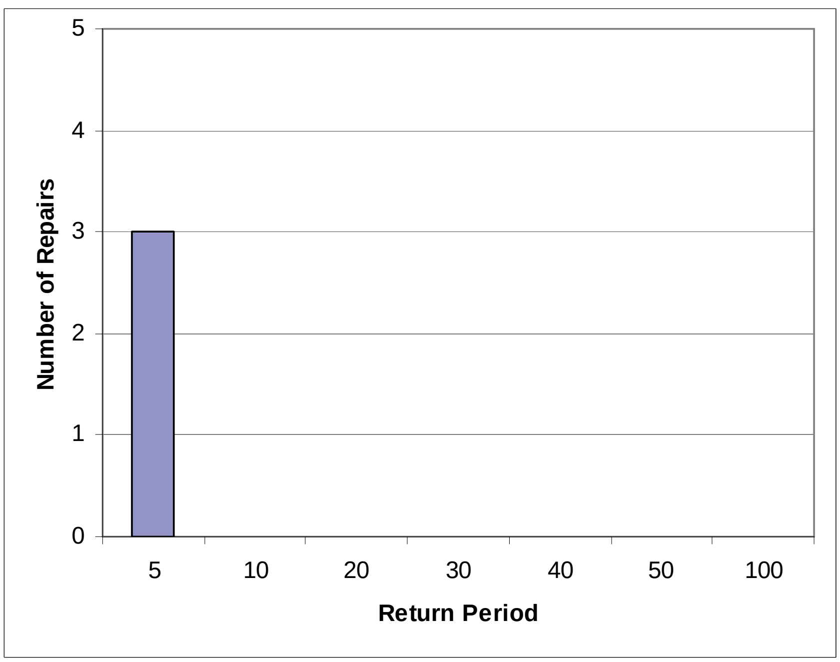


Figure 143. Total number of repairs and minor repairs due to armor damage as function of return period at sta 13 for armored crests

Summary and Recommendations

In general, there is no significant difference in cost between the least-cost armored crest section and that with an unarmored crest. Most optimal sections cost \$3,000/m to \$3,300/m (\$915/ft to \$1,006/ft). In general, the stations with more exposure (sta 1-8) demand higher return period designs (larger stone). These south- and west-facing reaches have minimum costs at 20- to 30-year return periods, while the more sheltered north- and east-facing stations (sta 10-13) have minimum costs at return periods of 5 to 10 years. Transition areas are near sta 2 and 9.

Hurricane Isabel produced maximum water levels of 1.68 m (5.5 ft). As such, a crest height of 1.83 m (6 ft) was considered here to be the absolute minimum allowable. Although the optimal armored cross section was set at this level, it may be desirable to set the crest height higher for additional safety.

Figures from the ELS analysis in this chapter show that the cross-sectional costs are generally not a strong function of return period for higher return period designs. There is little cost penalty in using a more reliable design with larger stone. Also, if conditions change, such as the rate of sea level rise, or if there is stone breakage or poor construction over a reach, then it will be desirable to have larger armor stones. Finally, all costs resulting from major sectional failure have not been included (e.g., environmental cleanup costs resulting from sediment contamination of oyster beds or SAV). Therefore, it is recommended that a conservative stone size be selected based on these figures; for the western reaches, the return period design of 40 years was used even though the 30-year return period appeared to be adequate. Similarly, for the more sheltered reaches, a larger stone size was selected. However, a third sectional design may be desirable for the very sheltered eastern reach.

The recommended sectional designs based on analysis of response to historical and simulated future wave climates are as follows:

Northeast to east reaches, sta 10-13

a. Design based on 30-year return period section at sta 10.

b. Crest: h_c = 2.13 m (7 ft) unarmored.

c. Armor stone weight: 350 lb.

d. Primary underlayer stone weight: 35 lb.

Southern, western, and northwest reaches, sta 1-9

a. Design based on 40-year return period section at sta 3.

b. Crest: h_c = 1.83 m (6 ft) armored.

c. Armor stone weight: 2,000 lb.

d. Primary underlayer stone weight: 200 lb.

For the upland cells, there is no armoring required for sta 10-13 if the roadway crest elevation of 2.13 m (7 ft) is employed. For upland cells at sta 7-9, significant armoring is required. A filter layer would be required under the armor for these stations, as well. The basic armor requirements are summarized as follows:

a. Station 7: Upland armor crest elevation = 3 m (10 ft), W_{a50} = 892 N (425 lb).

b. Station 8-9: Upland armor crest elevation = 2 m (7 ft), W_{a50} = 360 N (80 lb).

c. Station 10-13: No upland cell armoring is required.

9 Life-Cycle Simulation Results, Barren Island

This chapter describes the life-cycle structural optimization of Barren Island structures. Wave and water level results are presented in the following section. The methods used to develop these results are discussed in Chapter 5. Structure response and optimization are presented in the second section of this chapter. The methodology used to optimize the design of protective structures is given in Chapter 6. For Barren Island, only the historical wave climate has been used to analyze the structures.

Waves and Water Levels

The extremal H_s values for various return periods at the six Barren Island stations are shown in Figure 144. The results are tabulated for each station in Appendix H in order to provide more background information. Peak wave period and water depth are shown as functions of return period for the six stations in Figures 145 and 146, respectively. The figures indicate that the exposure does not vary much for the six stations. However, sta 1 and 6 have slightly reduced exposure. Station 1 is in very shallow water, and sta 6 has some sheltering from the more severe southern exposure.

Structural Optimization

Cross sections were analyzed as follows: (a) the southern portion opposite sta 1-3 consists of the low-crested offshore breakwater shown in Figure 147, and (b) the northern portion opposite sta 4-6 consists of the dike cross section shown in Figure 148.

The simple section shown in Figure 147 consists of a multi-layer section with two-stone-thick armor and underlayers. The crest is three stones wide. The toe is two stones wide and a single stone high. The side slopes are 1V:1.5H. The entire structure is underlain by a geotextile fabric. The layer thicknesses were assumed to be $2D_{a50}$ for armor, $2D_{u50}$ for filter layer, and D_{ta50} for toe armor. Here $D_{a50} = (V_{50})^{1/3} = (W_{50}/\gamma_r)^{1/3}$ is the nominal diameter of the armor weight corresponding to the 50 percent exceedance level on the weight distribution curve. Similarly, D_{u50} is the filter layer 50 percent exceedance nominal diameter and D_{ta50} is the toe armor 50 percent exceedance nominal diameter. The section shown in Figure 148 consists of the existing structure with a single layer overlay. The existing structure has a single layer of armor over a core. The existing toe is two stones wide by one stone high, similar to that described for sta 1-3. The new toe has the same dimensions and is placed seaward of the existing toe. The side slopes are 1V:1.5H. The entire structure rests on a geotextile fabric. On the lee side, the structure has a filter layer to prevent leakage of the sand within the fill.

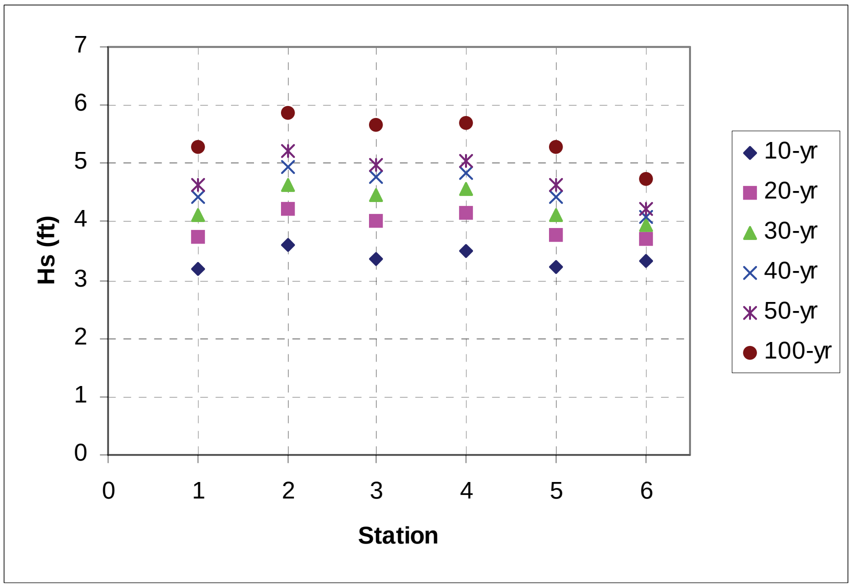


Figure 144. Return period H_s at nearshore stations, Barren Island

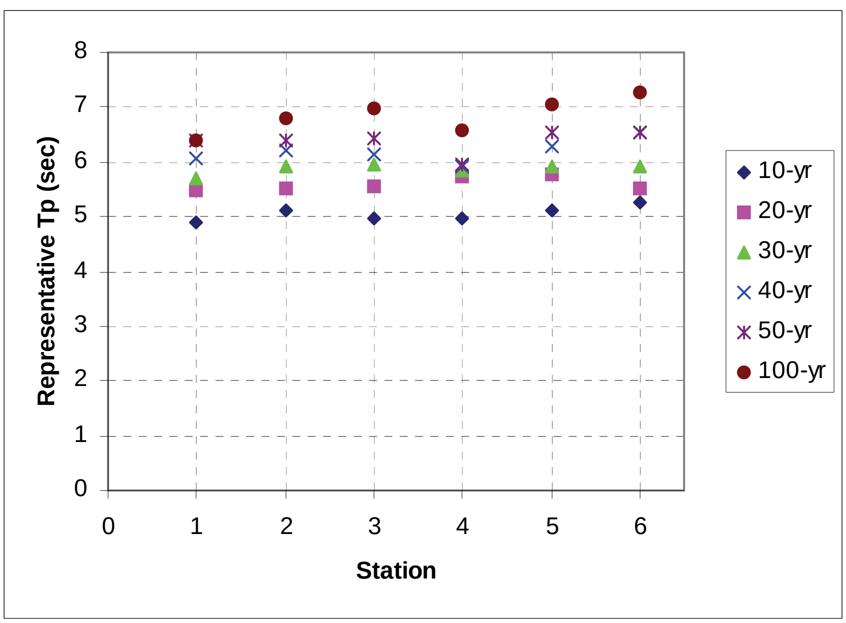


Figure 145. Return period T_p at nearshore stations, Barren Island

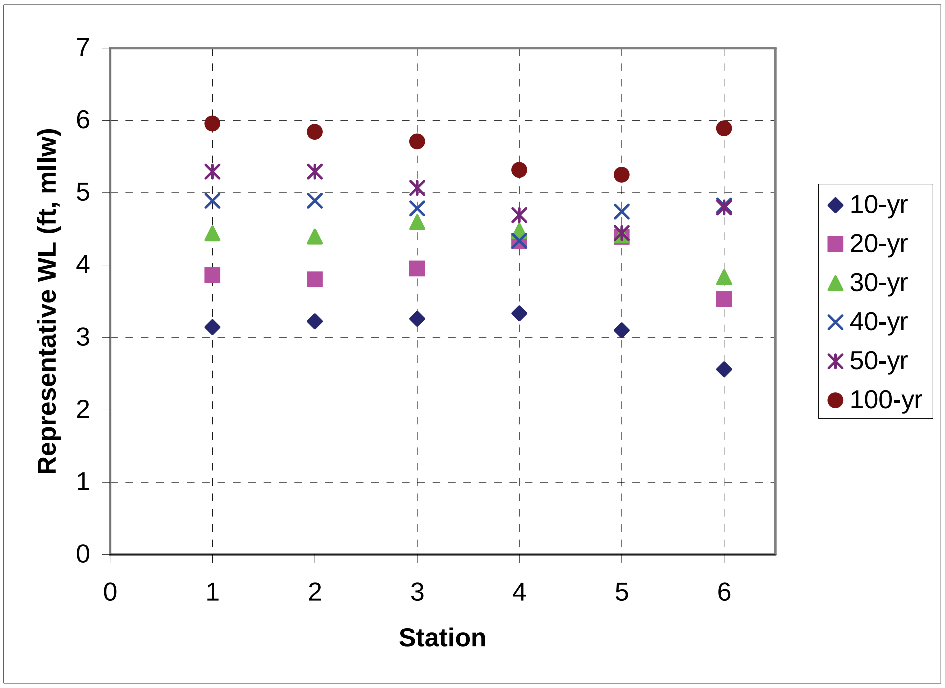


Figure 146. Return-period water level at nearshore stations, Barren Island

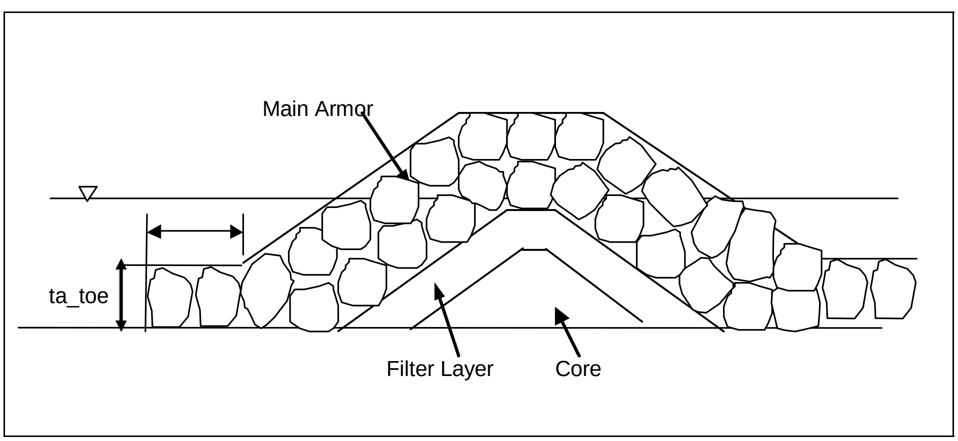


Figure 147. General cross section for sta 1, 2, and 3

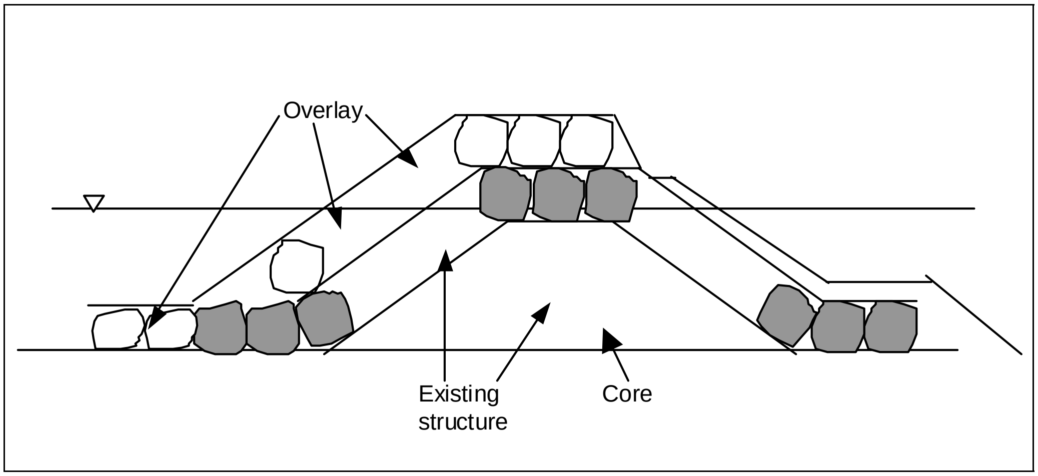


Figure 148. General cross section for sta 4, 5, and 6

Armor Stability

Values used to calculate armor stability are as follows:

a. Zero damage level: $S = 1$ was assumed.

b. Structure slope: 1V:1.5H was assumed for all sections.

c. Specific gravity: $S_r = 2.578$.

d. Number of waves for zero damage level: $N_z = 7{,}000$.

Appendix I summarizes the stable armor weight and stable underlayer weight as a function of return period for each design analysis station of Barren Island.

Summary and Recommendations

Stations 1-3

This section is a low-crested trapezoidal rubble mound breakwater structure for SAV protection as shown in Figure 147. In this case, a limiting wave height of $H_s = 1$ m (3 ft) was assumed for SAV protection. A crest height of 1.22 m (4 ft) provides SAV protection, limiting the wave height up to just over the 30-year return period storm event. A crest height of 1.83 m (6 ft) provides SAV protection for wave conditions exceeding the 50-year return period event. This result is based on an overtopping analysis of the structure and does not take into account wave transmission through the structure or diffraction through the gap between the mainland and the island. It also does not take into account local waves generated on the eastern side of the island. The recommended design is:

a. Return period = 50 years.

b. Alternative 1: Crest height = 1.22 m (4 ft).

(1) Main armor weight: W_{a50} = 4,448 N (1,000 lb).

(2) Underlayer weight: W_{u50} = 445 N (1,00 lb).

(3) Toe armor weight: W_{ta50} = 4,448 N (1,000 lb).

c. Alternative 2: Crest height of 1.83 m (6 ft).

(1) Main armor weight: W_{a50} = 7,117 N (1,600 lb).

(2) Underlayer weight: W_{u50} = 712 N (160 lb).

(3) Toe armor weight: W_{ta50} = 7,117 N (1,600 lb).

Stations 4-6

Low-crested trapezoidal rubble mound breakwater structure to retain fill as shown in Figure 148. Considering the rubble mound structure only, the structure stone size to prevent breaching is given in Appendix I for each return period. The cross section shows main armor across the crest. Therefore, the stability of the crest is roughly the same as the primary armor. The recommended design is:

a. Return period = 50 years.

b. Alternative 1: Crest height = 1.22 m (4 ft).

(1) Main armor weight: W_{a50} = 5,783 N (1,300 lb).

(2) Underlayer weight: W_{u50} = 578 N (130 lb).

(3) Toe armor weight: W_{ta50} = 5,783 (1,300 lb).

c. Alternative 2: Crest height of 1.83 m (6 ft).

(1) Main armor weight: W_{a50} = 7,117 N (1,600 lb).

(2) Underlayer weight: W_{u50} = 712 N (160 lb).

(3) Toe armor weight: W_{ta50} = 7,117 N (1,600 lb).

10 Summary and Conclusions

This report summarizes the life-cycle design and optimization of structures on three islands in Chesapeake Bay. The islands are Poplar, James, and Barren. The life-cycle analysis was accomplished using a new method termed Empirical Life-cycle Simulation (ELS). The entire analysis method as applied in this report can be summarized as follows:

a. Identify historical tropical and extratropical storms needed to develop design conditions at Chesapeake Bay project sites.

b. Acquire wind fields for these historical storms, to be used for water level modeling. Open-ocean winds for most storms were available from previous studies.

c. Adjust wind fields over Chesapeake Bay waters as needed to represent winds over the bay suitable for water level modeling.

d. Analyze existing historical data from regional anemometers to develop local winds over Chesapeake Bay fetches for wave analysis.

e. Compute historical storm water levels using the existing ADCIRC numerical model, updating the regional bathymetry and shoreline grid already developed for other Baltimore District studies at Ocean City Inlet and Assateague Island.

f. Hindcast historical storm waves using model winds along with measured winds from several area anemometers. Compute historical offshore waves using relationships for wind-wave growth over irregular, restricted fetches.

g. Transform waves through shallow nearshore waters to shore using a spectral wave transformation model (STWAVE).

h. Compute responses for these historical events, such as runup, overtopping as a function of crest height, structure damage as a function of stone size, and required toe stone weight. Use techniques based on recommendations given in the CEM.

i. Recreate multiple life cycles of storms and project responses using the ELS. Each life cycle represents a possible future condition that is statistically consistent with historical storm forcing, response, and sequencing information. The ELS simulation includes progressive revetment damage due to successive storms that may occur between maintenance opportunities. Realistic maintenance cycles are incorporated into the simulation. Compute life-cycle damage and function for selected designs that appear to be favorable.

j. Select optimal cross sections.

Candidate designs and design evaluation criteria, including environmental considerations, were defined in close coordination with the Baltimore District. The results are summarized based on analyses of mean and extreme structure responses in the multiple life-cycle scenarios. The results will assist the Baltimore District in quantifying design construction cost vs. benefit trade-offs between initial construction and expected maintenance.

The historical storms selected for simulation include both winter storms (extratropical storms) and hurricanes (tropical storms). The storms chosen, the reasons for choosing them, and the procedures for estimating storm wind and pressure fields are discussed in Chapter 2.

Storm wind and pressure fields over Chesapeake Bay provide the key meteorological forcing that can cause unusually high water levels during storms. To accurately simulate water levels caused by the combined effect of historical storms and astronomical tides, the entire Chesapeake Bay must be modeled. The procedures and results from these hydrodynamic simulations of historical storms are presented in Chapter 3.

Storm winds also generate unusually high waves. Adaptation of winds to local wave growth around the study islands, wave generation, and wave transformation to island shores are described in Chapter 4.

Generation of 148-year time histories of historical waves and water levels at specific nearshore locations around each structure for use in the life-cycle analysis phase of the study is described in Chapter 5. Also discussed is the empirical life-cycle simulation of waves and water levels. Finally, analysis of storm maximum waves and extremal analysis of waves and water levels is described. The results of the analyses are plotted and tabulated in Chapter 5. Appendices A, D, and G summarize storm maximum waves for Poplar, James, and Barren Islands, respectively. Appendices B, E, and H summarize extremal wave and water level conditions for Poplar, James and Barren Islands, respectively.

Chapter 6 provides the detailed structural analysis methods used throughout this study. Methods are discussed for determining rubble mound armor and toe stability and damage development on the revetment and breakwater structures. Methods and criteria for determining wave runup, wave overtopping volumetric transmission, and transmitted wave height are summarized. The structure life-cycle simulation and economic analysis are also described in Chapter 6.

Several structure types were analyzed. For the Poplar Island northern expansion, the primary structure is a rubble mound revetment or dike for containing clean dredge material, similar to the structure that surrounds the existing Poplar Island. A compound slope is specified to contain upland cells on the northeastern cells. A second alternative for Poplar, called the National Marine Fisheries Service Offshore Breakwater Alternative, is analyzed. This alternative includes a segmented offshore low-crested breakwater sheltering an embayment on the western exposed side of the island. A revetment armors the embayment along the shoreline. James Island includes a revetment containment structure with upland cells specified for the northern third of the island. This structure is similar to that already in use at Poplar Island. Barren Island structures include a containment dike and a low-crested breakwater for SAV protection.

The ELS methods used here proved to be very helpful in optimizing the structure cross sections. Cross sections were defined for eleven return-period wave and water level combinations. The cross sections were then exposed to historical and simulated wave and water level time series. Life-cycle damage and repair time histories resulting from armor instability were produced. Volumetric overtopping for revetments and wave transmission for breakwaters was determined. Toe damage was also determined. The analysis revealed the least-cost structure cross sections as well as the nature of damage to the structures. The number of minor and breach repairs due to armor instability, based on specified repair rules, were determined. Also, breach repairs resulting from overtopping failure of the revetment crests were quantified. The present-worth costs from all repairs were summed and added to the present-worth capital cost for each section to determine the overall life-cycle cost of each section. Life-cycle optimizations were completed for each of the three study islands. Poplar Island results are presented in Chapter 7, while results for James and Barren Islands are given in Chapters 8 and 9, respectively. The Barren Island structure design was analyzed using only historical waves and water levels because there is presently no damage model for the types of structure cross sections specified.

Although the results were clear and optimized sections determined, the study described here was for preliminary design. As such, some details were not resolved. The tall toe design was required to protect the sand dike during construction. However, it is costly, and an improved toe design is still sought. Toe stability equations often do not converge, so toe stone size estimates are sometimes crude. Overtopping volume failure criteria as given in design manuals are crude, and improved guidance is desired. Low-crested structure and toe stability design guidance is very limited. The equations for emergent and submerged structures are not continuous. Improved guidance for this project may be obtained from limited small-scale physical model studies and coordination with construction engineers.

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Appendix A
Maximum Significant Wave Height for Storm History for Poplar Island

Maximum significant wave height by storm, needed to determine return-period wave height values for structure design, was extracted along with corresponding peak wave period, wave direction, and water level. Separate output files were created for tropical storms only, northeasters only, and all storms together. These values of maximum H_s for each storm, as well as associated peak wave period, direction, and water level, are tabulated for all stations of Poplar Island in this Appendix. Tables A1-A7 summarize hurricanes for sta 33-39, respectively, and Tables A8-A14 summarize extratropical storms for Poplar Island.

Table A1
Maximum H_S by Storm, Poplar Island, Station 33, Tropical Storms

Storm Number	Storm Name	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.50 (1.64)	3.97	333.00	0.56 (1.84)	9
2	None	1861/09/28	0.32 (1.05)	2.23	273.96	1.00 (3.28)	6
3	None	1861/11/03	0.40 (1.31)	3.51	333.00	0.59 (1.94)	3
4	None	1863/09/19	0.49 (1.61)	3.93	333.00	0.56 (1.84)	3
5	None	1874/09/29	0.77 (2.53)	5.43	241.51	1.19 (3.90)	6
6	None	1876/09/18	0.99 (3.25)	5.97	241.65	1.61 (5.28)	12
7	None	1877/10/05	0.91 (2.99)	3.56	272.92	1.12 (3.67)	12
8	None	1878/10/23	2.26 (7.41)	8.45	243.64	2.22 (7.28)	15
9	None	1879/08/19	0.41 (1.35)	3.54	331.20	0.78 (2.56)	6
10	None	1880/09/10	0.27 (0.89)	2.79	335.81	0.71 (2.33)	0
11	None	1881/09/11	0.37 (1.21)	3.25	333.10	0.64 (2.10)	6
12	None	1888/10/12	0.15 (0.49)	2.78	221.43	0.30 (0.98)	0
13	None	1889/09/25	0.27 (0.89)	2.77	334.90	0.61 (2.00)	0
14	None	1893/06/17	0.37 (1.21)	3.33	333.00	0.40 (1.31)	6
15	None	1893/08/29	1.97 (6.46)	8.11	250.00	1.47 (4.82)	15
16	None	1893/10/14	0.58 (1.90)	4.50	328.89	1.34 (4.40)	6
17	None	1893/10/23	1.32 (4.33)	7.01	250.60	0.29 (0.95)	12
18	None	1894/09/29	0.55 (1.80)	4.22	327.00	0.42 (1.38)	24
19	None	1894/10/10	0.43 (1.41)	3.66	332.10	0.74 (2.43)	6
20	None	1897/10/25	0.52 (1.71)	4.11	327.00	0.32 (1.05)	12
21	None	1899/08/19	0.56 (1.84)	4.17	328.00	0.68 (2.23)	57
22	None	1899/11/01	1.48 (4.86)	7.09	246.62	1.62 (5.31)	9
23	None	1904/09/15	0.87 (2.85)	3.49	275.00	1.33 (4.36)	6
24	None	1908/08/01	0.38 (1.25)	3.42	333.00	0.62 (2.03)	12
25	None	1923/10/24	0.60 (1.97)	4.75	325.88	0.43 (1.41)	9
26	None	1933/08/24	0.99 (3.25)	5.98	240.30	1.61 (5.28)	24
27	None	1933/09/17	0.49 (1.61)	3.93	331.20	0.66 (2.17)	18
28	None	1935/09/06	0.25 (0.82)	2.68	335.81	0.55 (1.80)	0
29	None	1936/09/19	0.67 (2.20)	5.00	324.12	0.32 (1.05)	30
30	None	1944/08/03	0.53 (1.74)	2.76	331.99	1.08 (3.54)	6
31	None	1944/09/15	0.49 (1.61)	3.99	332.79	0.50 (1.64)	12
32	None	1946/07/07	0.16 (0.52)	2.26	333.00	0.47 (1.54)	0
33	Barbara	1953/08/14	0.46 (1.51)	3.89	331.00	0.54 (1.77)	12
34	Hazel	1954/10/16	2.47 (8.10)	8.82	243.64	2.19 (7.18)	12
35	Connie	1955/08/13	0.94 (3.08)	5.89	326.00	1.00 (3.28)	24
36	Diane	1955/08/18	0.78 (2.56)	5.48	249.00	0.99 (3.25)	18
37	Ione	1955/09/20	0.55 (1.80)	4.24	326.12	0.32 (1.05)	12
38	Brenda	1960/07/30	0.32 (1.05)	2.96	337.72	0.86 (2.82)	3
39	Donna	1960/09/12	0.57 (1.87)	4.24	329.77	0.75 (2.46)	9
40	Doria	1967/09/12	0.02 (0.07)	0.57	332.96	0.62 (2.03)	0
41	Doria	1971/08/28	0.51 (1.67)	3.89	333.10	0.95 (3.12)	3
42	Bret	1981/07/01	0.37 (1.21)	3.31	334.80	0.28 (0.92)	6
43	Dean	1983/09/30	0.12 (0.39)	1.95	333.90	0.43 (1.41)	0
44	Gloria	1985/09/27	0.86 (2.82)	5.73	323.12	0.50 (1.64)	9
45	Charley	1986/08/18	0.44 (1.44)	3.80	331.89	0.45 (1.48)	12
46	Danielle	1992/09/26	0.23 (0.75)	2.61	333.90	0.54 (1.77)	0
47	Bertha	1996/07/13	0.61 (2.00)	2.97	275.00	1.00 (3.28)	3
48	Fran	1996/09/07	0.40 (1.31)	4.00	229.12	1.25 (4.10)	3
49	Bonnie	1998/08/28	0.35 (1.15)	3.14	334.90	0.75 (2.46)	9
50	Earl	1998/09/05	0.14 (0.46)	2.74	221.43	0.36 (1.18)	0
51	Floyd	1999/09/17	0.59 (1.94)	4.49	329.89	1.06 (3.48)	9
52	Isabel	2003/09/19	1.01 (3.31)	5.92	228.13	2.12 (6.96)	15

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table A2
Maximum H_s by Storm, Poplar Island, Station 34, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.48 (1.57)	3.97	330.00	0.56 (1.84)	9
2	None	1861/09/28	0.34 (1.12)	4.08	247.25	0.92 (3.02)	6
3	None	1861/11/03	0.38 (1.25)	3.51	330.00	0.59 (1.94)	3
4	None	1863/09/19	0.47 (1.54)	3.93	330.00	0.56 (1.84)	6
5	None	1874/0929	0.92 (3.02)	5.43	242.49	1.19 (3.90)	12
6	None	1876/09/18	1.17 (3.84)	5.97	244.63	1.61 (5.28)	12
7	None	1877/10/05	0.89 (2.92)	3.56	272.92	1.12 (3.67)	12
8	None	1878/10/23	2.54 (8.33)	8.45	244.63	2.22 (7.28)	15
9	None	1879/08/19	0.39 (1.28)	3.54	328.22	0.78 (2.56)	6
10	None	1880/09/10	0.26 (0.85)	2.79	332.79	0.71 (2.33)	0
11	None	1881/09/11	0.35 (1.15)	3.25	330.11	0.64 (2.10)	3
12	None	1888/10/12	0.17 (0.56)	2.78	234.58	0.30 (0.98)	0
13	None	1889/09/25	0.26 (0.85)	2.77	331.89	0.61 (2.00)	0
14	None	1893/06/17	0.36 (1.18)	3.33	330.00	0.40 (1.31)	6
15	None	1893/08/29	2.06 (6.76)	8.11	250.00	1.47 (4.82)	18
16	None	1893/10/14	0.54 (1.77)	4.32	244.00	1.13 (3.71)	6
17	None	1893/10/23	1.14 (3.74)	6.61	249.61	0.29 (0.95)	15
18	None	1894/09/29	0.51 (1.67)	4.22	324.00	0.42 (1.38)	24
19	None	1894/10/10	0.41 (1.35)	3.66	329.11	0.74 (2.43)	6
20	None	1897/10/25	0.49 (1.61)	4.11	324.00	0.32 (1.05)	12
21	None	1899/08/19	0.53 (1.74)	4.17	326.00	0.68 (2.23)	57
22	None	1899/11/01	1.67 (5.48)	7.09	246.62	1.62 (5.31)	12
23	None	1904/09/15	0.84 (2.76)	3.49	275.00	1.33 (4.36)	9
24	None	1908/08/01	0.36 (1.18)	3.42	330.00	0.62 (2.03)	12
25	None	1923/10/24	0.52 (1.71)	4.75	322.87	0.43 (1.41)	9
26	None	1933/08/24	1.17 (3.84)	5.98	243.27	1.61 (5.28)	24
27	None	1933/09/17	0.47 (1.54)	3.93	328.22	0.66 (2.17)	18
28	None	1935/09/06	0.24 (0.79)	2.68	332.79	0.55 (1.80)	0
29	None	1936/09/19	0.59 (1.94)	5.00	321.13	0.32 (1.05)	30
30	None	1944/08/03	0.53 (1.74)	2.76	330.99	1.08 (3.54)	6
31	None	1944/09/15	0.46 (1.51)	3.99	329.77	0.50 (1.64)	9
32	None	1946/07/07	0.16 (0.52)	2.26	330.00	0.47 (1.54)	0
33	Barbara	1953/08/14	0.44 (1.44)	3.89	328.00	0.54 (1.77)	12
34	Hazel	1954/10/16	2.75 (9.02)	8.82	244.63	2.19 (7.18)	12
35	Connie	1955/08/13	0.81 (2.66)	5.89	323.00	1.00 (3.28)	27
36	Diane	1955/08/18	0.89 (2.92)	5.48	249.00	0.99 (3.25)	21
37	Ione	1955/09/20	0.51 (1.67)	4.24	323.12	0.32 (1.05)	12
38	Brenda	1960/07/30	0.33 (1.08)	3.66	226.20	0.79 (2.59)	6
39	Donna	1960/09/12	0.54 (1.77)	4.24	327.76	0.75 (2.46)	9
40	Doria	1967/09/12	0.02 (0.07)	0.57	332.96	0.62 (2.03)	0
41	Doria	1971/08/28	0.49 (1.61)	3.89	330.11	0.95 (3.12)	3
42	Bret	1981/07/01	0.35 (1.15)	3.31	331.78	0.28 (0.92)	6
43	Dean	1983/09/30	0.12 (0.39)	1.95	330.89	0.43 (1.41)	0
44	Gloria	1985/09/27	0.72 (2.36)	5.73	320.13	0.50 (1.64)	9
45	Charley	1986/08/18	0.42 (1.38)	3.80	328.89	0.45 (1.48)	12
46	Danielle	1992/09/26	0.22 (0.72)	2.61	330.89	0.54 (1.77)	0
47	Bertha	1996/07/13	0.59 (1.94)	2.97	275.00	1.00 (3.28)	3
48	Fran	1996/09/07	0.46 (1.51)	4.00	236.98	1.25 (4.10)	3
49	Bonnie	1998/08/28	0.33 (1.08)	3.08	332.90	0.91 (2.99)	9
50	Earl	1998/09/05	0.22 (0.72)	3.02	225.48	0.56 (1.84)	0
51	Floyd	1999/09/17	0.56 (1.84)	4.49	327.88	1.06 (3.48)	6
52	Isabel	2003/09/19	1.20 (3.94)	5.92	236.00	2.12 (6.96)	12

[1]Storm duration is the time during a storm when H_S > 0.3 m.

Table A3
Maximum H_S by Storm, Poplar Island, Station 35, Tropical Storms

Storm Number	Storm Name	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.50 (1.64)	2.62	40.02	0.42 (1.38)	15
2	None	1861/09/26	0.01 (0.03)	0.48	229.12	0.29 (0.95)	0
3	None	1861/11/03	0.57 (1.87)	2.78	40.02	0.55 (1.80)	9
4	None	1863/09/19	0.44 (1.44)	3.93	330.00	0.57 (1.87)	3
5	None	1874/09/28	0.01 (0.03)	1.23	230.41	0.29 (0.95)	0
6	None	1876/09/17	0.01 (0.03)	1.21	227.82	0.29 (0.95)	0
7	None	1877/10/03	0.01 (0.03)	0.93	227.82	0.29 (0.95)	0
8	None	1878/10/22	0.01 (0.03)	1.17	229.12	0.29 (0.95)	0
9	None	1879/08/19	1.01 (3.31)	3.62	44.00	0.85 (2.79)	6
10	None	1880/09/09	0.39 (1.28)	2.33	45.95	0.60 (1.97)	3
11	None	1881/09/10	0.45 (1.48)	2.48	43.00	0.72 (2.36)	9
12	None	1888/10/12	0.26 (0.85)	1.90	41.98	0.49 (1.61)	0
13	None	1889/09/25	0.36 (1.18)	2.23	43.98	0.80 (2.62)	3
14	None	1893/06/17	0.37 (1.21)	2.26	43.98	0.63 (2.07)	6
15	None	1893/08/27	0.15 (0.49)	1.44	42.96	0.29 (0.95)	0
16	None	1893/10/14	0.75 (2.46)	3.14	45.01	1.38 (4.53)	6
17	None	1893/10/21	0.11 (0.36)	1.28	40.02	0.29 (0.95)	0
18	None	1894/09/29	0.47 (1.54)	4.22	326.00	0.43 (1.41)	24
19	None	1894/10/10	0.68 (2.23)	3.02	43.98	0.69 (2.26)	6
20	None	1897/10/25	0.45 (1.48)	4.11	326.00	0.33 (1.08)	9
21	None	1899/08/19	0.49 (1.61)	4.17	327.00	0.69 (2.26)	57
22	None	1899/10/31	0.17 (0.56)	1.58	43.93	0.31 (1.02)	0
23	None	1904/09/14	0.21 (0.69)	1.75	41.95	0.25 (0.82)	0
24	None	1908/08/01	0.37 (1.21)	2.26	40.02	0.50 (1.64)	15
25	None	1923/10/24	0.48 (1.57)	4.75	324.88	0.44 (1.44)	6
26	None	1933/08/24	0.92 (3.02)	3.49	44.00	0.82 (2.69)	15
27	None	1933/09/16	0.61 (2.00)	2.87	42.02	0.63 (2.07)	27
28	None	1935/09/06	0.23 (0.75)	2.68	332.79	0.56 (1.84)	0
29	None	1936/09/19	0.55 (1.80)	2.79	330.99	0.31 (1.02)	30
30	None	1944/08/03	0.53 (1.74)	2.76	331.99	1.08 (3.54)	3
31	None	1944/09/14	0.66 (2.17)	2.99	43.98	0.59 (1.94)	18
32	None	1946/07/07	0.27 (0.89)	1.92	41.00	0.43 (1.41)	0
33	Barbara	1953/08/14	0.53 (1.74)	2.69	42.02	0.68 (2.23)	21
34	Hazel	1954/10/15	0.44 (1.44)	2.46	43.93	0.28 (0.92)	6
35	Connie	1955/08/13	0.74 (2.43)	5.89	324.00	1.00 (3.28)	33
36	Diane	1955/08/17	0.31 (1.02)	1.99	44.97	0.57 (1.87)	3
37	Ione	1955/09/19	0.65 (2.13)	2.97	43.00	0.54 (1.77)	39
38	Brenda	1960/07/30	0.30 (0.98)	2.96	334.69	0.86 (2.82)	0
39	Donna	1960/09/12	0.70 (2.30)	3.04	48.97	0.83 (2.72)	9
40	Doria	1967/09/11	0.06 (0.20)	0.96	41.98	0.29 (0.95)	0
41	Doria	1971/08/28	0.77 (2.53)	3.17	47.98	0.84 (2.76)	9
42	Bret	1981/07/01	0.80 (2.62)	3.25	41.98	0.33 (1.08)	12
43	Dean	1983/09/30	0.11 (0.36)	1.95	330.89	0.43 (1.41)	0
44	Gloria	1985/09/27	1.10 (3.61)	3.77	41.02	0.64 (2.10)	24
45	Charley	1986/08/18	0.56 (1.84)	2.76	42.02	0.68 (2.23)	21
46	Danielle	1992/09/26	0.32 (1.05)	2.12	41.00	0.35 (1.15)	3
47	Bertha	1996/07/13	1.03 (3.38)	3.65	44.00	1.06 (3.48)	3
48	Fran	1996/09/06	0.54 (1.77)	2.70	45.95	0.58 (1.90)	15
49	Bonnie	1998/08/28	0.46 (1.51)	2.48	46.00	0.94 (3.08)	42
50	Earl	1998/09/02	0.01 (0.03)	0.78	233.00	0.29 (0.95)	0
51	Floyd	1999/09/16	1.03 (3.38)	3.64	45.97	1.15 (3.77)	6
52	Isabel	2003/09/19	1.15 (3.77)	3.83	45.97	0.93 (3.05)	9

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table A4
Maximum H_s by Storm, Poplar Island, Station 36, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.51 (1.67)	2.62	41.02	0.42 (1.38)	15
2	None	1861/09/26	0.01 (0.03)	0.48	226.17	0.29 (0.95)	0
3	None	1861/11/03	0.58 (1.90)	2.78	41.02	0.55 (1.80)	9
4	None	1863/09/19	0.43 (1.41)	3.93	329.00	0.57 (1.87)	3
5	None	1874/09/28	0.01 (0.03)	1.23	227.44	0.29 (0.95)	0
6	None	1876/09/17	0.01 (0.03)	1.21	224.89	0.29 (0.95)	0
7	None	1877/10/03	0.01 (0.03)	0.93	224.89	0.29 (0.95)	0
8	None	1878/10/22	0.01 (0.03)	1.17	226.17	0.29 (0.95)	0
9	None	1879/08/19	1.04 (3.41)	3.62	44.00	0.85 (2.79)	6
10	None	1880/09/09	0.40 (1.31)	2.33	46.96	0.60 (1.97)	3
11	None	1881/09/10	0.46 (1.51)	2.48	44.00	0.72 (2.36)	9
12	None	1888/10/12	0.26 (0.85)	1.90	42.98	0.49 (1.61)	0
13	None	1889/09/25	0.36 (1.18)	2.23	44.99	0.80 (2.62)	3
14	None	1893/06/17	0.37 (1.21)	2.26	44.99	0.63 (2.07)	6
15	None	1893/08/27	0.15 (0.49)	1.44	43.96	0.29 (0.95)	0
16	None	1893/10/14	0.77 (2.53)	3.14	45.01	1.38 (4.53)	6
17	None	1893/10/21	0.11 (0.36)	1.28	41.02	0.29 (0.95)	0
18	None	1894/09/29	0.45 (1.48)	4.22	326.00	0.43 (1.41)	24
19	None	1894/10/10	0.69 (2.26)	3.02	44.99	0.69 (2.26)	6
20	None	1897/10/25	0.43 (1.41)	4.11	326.00	0.33 (1.08)	9
21	None	1899/08/19	0.48 (1.57)	4.17	327.00	0.69 (2.26)	54
22	None	1899/10/31	0.18 (0.59)	1.58	44.94	0.31 (1.02)	0
23	None	1904/09/14	0.22 (0.72)	1.75	42.96	0.25 (0.82)	0
24	None	1908/08/01	0.37 (1.21)	2.26	41.02	0.50 (1.64)	15
25	None	1923/10/24	0.47 (1.54)	4.75	324.88	0.44 (1.44)	6
26	None	1933/08/24	0.95 (3.12)	3.49	44.00	0.82 (2.69)	12
27	None	1933/09/16	0.62 (2.03)	2.87	43.01	0.63 (2.07)	27
28	None	1935/09/06	0.22 (0.72)	2.68	331.78	0.56 (1.84)	0
29	None	1936/09/19	0.54 (1.77)	2.79	330.99	0.31 (1.02)	30
30	None	1944/08/03	0.53 (1.74)	2.76	331.99	1.08 (3.54)	3
31	None	1944/09/14	0.67 (2.20)	2.99	44.99	0.59 (1.94)	18
32	None	1946/07/07	0.27 (0.89)	1.92	42.00	0.43 (1.41)	0
33	Barbara	1953/08/14	0.54 (1.77)	2.69	43.01	0.68 (2.23)	24
34	Hazel	1954/10/15	0.45 (1.48)	2.46	44.94	0.28 (0.92)	6
35	Connie	1955/08/13	0.71 (2.33)	5.89	324.00	1.00 (3.28)	33
36	Diane	1955/08/17	0.32 (1.05)	1.99	45.97	0.57 (1.87)	3
37	Ione	1955/09/19	0.66 (2.17)	2.97	44.00	0.54 (1.77)	39
38	Brenda	1960/07/30	0.29 (0.95)	2.96	333.68	0.86 (2.82)	0
39	Donna	1960/09/12	0.72 (2.36)	3.04	48.97	0.83 (2.72)	9
40	Doria	1967/09/11	0.06 (0.20)	0.96	42.98	0.29 (0.95)	0
41	Doria	1971/08/28	0.79 (2.59)	3.17	47.98	0.84 (2.76)	9
42	Bret	1981/07/01	0.83 (2.72)	3.25	42.98	0.33 (1.08)	9
43	Dean	1983/09/30	0.10 (0.33)	1.95	329.89	0.43 (1.41)	0
44	Gloria	1985/09/27	1.15 (3.77)	3.77	41.02	0.64 (2.10)	24
45	Charley	1986/08/18	0.57 (1.87)	2.76	43.01	0.68 (2.23)	21
46	Danielle	1992/09/26	0.33 (1.08)	2.12	42.00	0.35 (1.15)	3
47	Bertha	1996/07/13	1.06 (3.48)	3.65	44.00	1.06 (3.48)	3
48	Fran	1996/09/06	0.55 (1.80)	2.70	46.96	0.58 (1.90)	15
49	Bonnie	1998/08/28	0.47 (1.54)	2.48	46.00	0.94 (3.08)	45
50	Earl	1998/09/02	0.01 (0.03)	0.78	230.00	0.29 (0.95)	0
51	Floyd	1999/09/16	1.06 (3.48)	3.64	45.97	1.15 (3.77)	6
52	Isabel	2003/09/19	1.19 (3.90)	3.83	45.97	0.93 (3.05)	9

[1]Storm duration is the time during a storm when H_s > 0.3 m.

Table A5
Maximum H_S by Storm, Poplar Island, Station 37, Tropical Storms

Storm Number	Storm Name	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.39 (1.28)	2.62	45.01	0.42 (1.38)	15
2	None	1861/09/26	0.01 (0.03)	0.48	191.75	0.29 (0.95)	0
3	None	1861/11/03	0.44 (1.44)	2.78	45.01	0.55 (1.80)	6
4	None	1863/09/17	0.01 (0.03)	0.49	186.23	0.29 (0.95)	0
5	None	1874/09/28	0.01 (0.03)	1.23	192.83	0.29 (0.95)	0
6	None	1876/09/17	0.01 (0.03)	1.21	190.67	0.29 (0.95)	0
7	None	1877/10/03	0.01 (0.03)	0.93	190.67	0.29 (0.95)	0
8	None	1878/10/22	0.01 (0.03)	1.17	191.75	0.29 (0.95)	0
9	None	1879/08/19	0.79 (2.59)	3.62	45.00	0.85 (2.79)	3
10	None	1880/09/09	0.32 (1.05)	2.33	49.98	0.60 (1.97)	3
11	None	1881/09/10	0.36 (1.18)	2.48	47.00	0.72 (2.36)	24
12	None	1888/10/12	0.20 (0.66)	1.90	46.99	0.49 (1.61)	3
13	None	1889/09/25	0.29 (0.95)	2.23	47.99	0.80 (2.62)	3
14	None	1893/06/17	0.30 (0.98)	2.26	47.99	0.63 (2.07)	3
15	None	1893/08/27	0.11 (0.36)	1.44	47.98	0.29 (0.95)	0
16	None	1893/10/14	0.63 (2.07)	3.14	48.00	1.38 (4.53)	3
17	None	1893/10/21	0.08 (0.26)	1.28	45.01	0.29 (0.95)	0
18	None	1894/09/29	0.33 (1.08)	2.39	46.01	0.63 (2.07)	12
19	None	1894/10/10	0.55 (1.80)	3.02	47.99	0.69 (2.26)	3
20	None	1897/10/25	0.10 (0.33)	1.35	46.00	0.47 (1.54)	0
21	None	1899/08/17	0.27 (0.89)	2.20	46.00	0.45 (1.48)	21
22	None	1899/10/31	0.13 (0.43)	1.58	48.97	0.31 (1.02)	0
23	None	1904/09/14	0.16 (0.52)	1.71	47.98	0.35 (1.15)	3
24	None	1908/08/01	0.28 (0.92)	2.26	45.01	0.50 (1.64)	30
25	None	1923/10/22	0.04 (0.13)	0.95	45.01	0.37 (1.21)	0
26	None	1933/08/24	0.73 (2.39)	3.49	45.00	0.82 (2.69)	3
27	None	1933/09/16	0.49 (1.61)	2.87	46.01	0.63 (2.07)	12
28	None	1935/09/04	0.01 (0.03)	0.88	185.00	0.29 (0.95)	0
29	None	1936/09/17	0.01 (0.03)	1.65	337.91	0.29 (0.95)	0
30	None	1944/08/01	0.10 (0.33)	1.36	47.98	0.30 (0.98)	0
31	None	1944/09/14	0.53 (1.74)	2.99	47.99	0.59 (1.94)	27
32	None	1946/07/07	0.20 (0.66)	1.92	46.00	0.43 (1.41)	3
33	Barbara	1953/08/14	0.43 (1.41)	2.69	46.01	0.68 (2.23)	24
34	Hazel	1954/10/15	0.34 (1.12)	2.46	48.97	0.28 (0.92)	12
35	Connie	1955/08/12	0.50 (1.64)	2.97	45.01	0.44 (1.44)	36
36	Diane	1955/08/17	0.25 (0.82)	1.99	48.99	0.57 (1.87)	18
37	Ione	1955/09/19	0.52 (1.71)	2.97	47.00	0.54 (1.77)	33
38	Brenda	1960/07/28	0.01 (0.03)	0.54	195.00	0.29 (0.95)	0
39	Donna	1960/09/12	0.59 (1.94)	3.04	51.99	0.83 (2.72)	3
40	Doria	1967/09/11	0.04 (0.13)	0.96	46.99	0.29 (0.95)	0
41	Doria	1971/08/28	0.64 (2.10)	3.17	51.00	0.84 (2.76)	6
42	Bret	1981/07/01	0.61 (2.00)	3.25	46.99	0.33 (1.08)	6
43	Dean	1983/09/28	0.01 (0.03)	0.54	46.99	0.34 (1.12)	0
44	Gloria	1985/09/27	0.83 (2.72)	3.77	44.01	0.64 (2.10)	24
45	Charley	1986/08/18	0.45 (1.48)	2.76	46.01	0.68 (2.23)	12
46	Danielle	1992/09/26	0.25 (0.82)	2.12	46.00	0.35 (1.15)	12
47	Bertha	1996/07/13	0.81 (2.66)	3.65	45.00	1.06 (3.48)	3
48	Fran	1996/09/06	0.44 (1.44)	2.70	49.98	0.58 (1.90)	30
49	Bonnie	1998/08/28	0.39 (1.28)	2.48	49.00	0.94 (3.08)	54
50	Earl	1998/09/02	0.01 (0.03)	0.78	195.00	0.29 (0.95)	0
51	Floyd	1999/09/16	0.81 (2.66)	3.64	46.98	1.15 (3.77)	27
52	Isabel	2003/09/19	0.89 (2.92)	3.83	46.98	0.93 (3.05)	9

[1]Storm duration is the time during a storm when H_S > 0.15 m.

Table A6
Maximum H_s by Storm, Poplar Island, Station 38, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.30 (0.98)	2.62	36.03	0.44 (1.44)	9
2	None	1861/09/28	0.16 (0.52)	4.08	143.89	0.92 (3.02)	6
3	None	1861/11/03	0.34 (1.12)	2.78	36.03	0.56 (1.84)	6
4	None	1863/09/19	0.13 (0.43)	3.04	148.00	0.71 (2.33)	0
5	None	1874/09/29	0.19 (0.62)	5.43	139.82	1.17 (3.84)	9
6	None	1876/09/18	0.28 (0.92)	5.97	144.19	1.63 (5.35)	15
7	None	1877/10/05	0.18 (0.59)	4.27	143.89	0.99 (3.25)	3
8	None	1878/10/23	0.74 (2.43)	8.45	141.21	2.30 (7.55)	21
9	None	1879/08/19	0.64 (2.10)	3.62	40.00	0.86 (2.82)	3
10	None	1880/09/09	0.26 (0.85)	2.33	44.94	0.60 (1.97)	3
11	None	1881/09/10	0.30 (0.98)	2.48	42.00	0.72 (2.36)	12
12	None	1888/10/12	0.15 (0.49)	1.90	37.97	0.49 (1.61)	0
13	None	1889/09/25	0.24 (0.79)	2.23	42.98	0.80 (2.62)	3
14	None	1893/06/17	0.24 (0.79)	2.26	42.98	0.63 (2.07)	3
15	None	1893/08/29	0.40 (1.31)	8.11	138.00	1.44 (4.72)	15
16	None	1893/10/14	0.55 (1.80)	3.14	47.00	1.38 (4.53)	9
17	None	1893/10/23	0.17 (0.56)	7.01	134.25	0.31 (1.02)	3
18	None	1894/09/29	0.27 (0.89)	2.39	41.02	0.63 (2.07)	6
19	None	1894/10/10	0.45 (1.48)	3.02	42.98	0.69 (2.26)	3
20	None	1897/10/25	0.08 (0.26)	1.35	37.00	0.47 (1.54)	0
21	None	1899/08/17	0.21 (0.69)	2.20	37.00	0.45 (1.48)	12
22	None	1899/11/01	0.37 (1.21)	7.09	140.22	1.61 (5.28)	12
23	None	1904/09/15	0.20 (0.66)	3.66	154.53	1.27 (4.17)	9
24	None	1908/08/01	0.23 (0.75)	2.20	41.02	0.64 (2.10)	21
25	None	1923/10/24	0.12 (0.39)	3.06	149.45	0.36 (1.18)	0
26	None	1933/08/24	0.59 (1.94)	3.49	40.00	0.85 (2.79)	15
27	None	1933/09/16	0.40 (1.31)	2.87	41.02	0.64 (2.10)	12
28	None	1935/09/06	0.08 (0.26)	2.59	143.73	0.33 (1.08)	0
29	None	1936/09/17	0.01 (0.03)	1.65	357.96	0.29 (0.95)	0
30	None	1944/08/03	0.19 (0.62)	3.98	145.83	1.10 (3.61)	9
31	None	1944/09/14	0.44 (1.44)	2.99	42.98	0.61 (2.00)	18
32	None	1946/07/07	0.16 (0.52)	1.92	37.00	0.43 (1.41)	3
33	Barbara	1953/08/14	0.35 (1.15)	2.69	41.02	0.68 (2.23)	18
34	Hazel	1954/10/16	0.79 (2.59)	8.82	141.21	2.30 (7.55)	27
35	Connie	1955/08/12	0.38 (1.25)	2.97	36.03	0.45 (1.48)	30
36	Diane	1955/08/17	0.21 (0.69)	1.99	43.96	0.56 (1.84)	27
37	Ione	1955/09/19	0.43 (1.41)	2.97	42.00	0.56 (1.84)	30
38	Brenda	1960/07/30	0.14 (0.46)	3.38	143.73	0.46 (1.51)	0
39	Donna	1960/09/12	0.52 (1.71)	3.04	50.99	0.84 (2.76)	3
40	Doria	1967/09/11	0.03 (0.10)	0.96	37.97	0.29 (0.95)	0
41	Doria	1971/08/28	0.56 (1.84)	3.17	49.99	0.85 (2.79)	6
42	Bret	1981/07/01	0.48 (1.57)	3.25	37.97	0.35 (1.15)	6
43	Dean	1983/09/28	0.01 (0.03)	0.54	37.97	0.34 (1.12)	0
44	Gloria	1985/09/27	0.67 (2.20)	3.77	37.03	0.69 (2.26)	24
45	Charley	1986/08/18	0.37 (1.21)	2.76	41.02	0.68 (2.23)	9
46	Danielle	1992/09/26	0.19 (0.62)	2.12	37.00	0.35 (1.15)	3
47	Bertha	1996/07/13	0.65 (2.13)	3.65	40.00	1.06 (3.48)	3
48	Fran	1996/09/06	0.36 (1.18)	2.70	44.94	0.60 (1.97)	30
49	Bonnie	1998/08/28	0.34 (1.12)	2.48	48.00	0.95 (3.12)	48
50	Earl	1998/09/05	0.12 (0.39)	3.02	149.64	0.55 (1.80)	0
51	Floyd	1999/09/16	0.65 (2.13)	3.64	41.95	1.18 (3.87)	9
52	Isabel	2003/09/19	0.72 (2.36)	3.83	41.95	0.97 (3.18)	24

[1] Storm duration is the time during a storm when $H_S > 0.15$ m.

Table A7
Maximum H_S by Storm, Poplar Island, Station 39, Tropical Storms

Storm Number	Storm Name	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/19	0.04 (0.13)	1.48	151.00	0.30 (0.98)	0
2	None	1861/09/28	0.34 (1.12)	4.08	152.00	0.92 (3.02)	18
3	None	1861/11/02	0.13 (0.43)	2.69	153.00	0.66 (2.17)	0
4	None	1863/09/19	0.24 (0.79)	3.61	153.85	0.62 (2.03)	9
5	None	1874/09/29	0.38 (1.25)	5.43	148.62	1.18 (3.87)	21
6	None	1876/09/18	0.55 (1.80)	5.97	153.14	1.64 (5.38)	24
7	None	1877/10/05	0.38 (1.25)	4.27	152.00	1.00 (3.28)	9
8	None	1878/10/23	1.44 (4.72)	8.45	152.15	2.33 (7.64)	27
9	None	1879/08/18	0.22 (0.72)	3.38	155.00	0.88 (2.89)	3
10	None	1880/09/09	0.18 (0.59)	2.67	147.34	0.50 (1.64)	3
11	None	1881/09/10	0.09 (0.30)	2.26	153.00	0.59 (1.94)	0
12	None	1888/10/12	0.19 (0.62)	2.78	150.66	0.31 (1.02)	12
13	None	1889/09/24	0.23 (0.75)	3.01	147.34	0.51 (1.67)	15
14	None	1893/06/17	0.10 (0.33)	2.08	144.03	0.44 (1.44)	0
15	None	1893/08/29	0.87 (2.85)	8.11	149.00	1.40 (4.59)	27
16	None	1893/10/14	0.44 (1.44)	4.32	149.00	1.14 (3.74)	9
17	None	1893/10/23	0.43 (1.41)	7.01	140.22	0.32 (1.05)	24
18	None	1894/09/28	0.06 (0.20)	1.81	154.02	0.55 (1.80)	0
19	None	1894/10/10	0.21 (0.69)	2.92	149.60	0.59 (1.94)	12
20	None	1897/10/24	0.02 (0.07)	1.12	152.00	0.44 (1.44)	0
21	None	1899/08/15	0.04 (0.13)	1.51	149.00	0.40 (1.31)	0
22	None	1899/11/01	0.77 (2.53)	7.09	151.16	1.62 (5.31)	18
23	None	1904/09/15	0.36 (1.18)	4.58	154.03	1.19 (3.90)	18
24	None	1908/07/30	0.04 (0.13)	1.51	149.00	0.30 (0.98)	0
25	None	1923/10/24	0.24 (0.79)	3.06	151.48	0.35 (1.15)	6
26	None	1933/08/24	0.55 (1.80)	5.98	152.29	1.63 (5.35)	15
27	None	1933/09/14	0.01 (0.03)	1.58	12.00	0.29 (0.95)	0
28	None	1935/09/06	0.17 (0.56)	2.59	145.69	0.34 (1.12)	6
29	None	1936/09/17	0.01 (0.03)	1.65	12.00	0.29 (0.95)	0
30	None	1944/08/03	0.36 (1.18)	3.98	150.69	1.11 (3.64)	15
31	None	1944/09/15	0.14 (0.46)	2.41	150.66	0.46 (1.51)	0
32	None	1946/07/05	0.05 (0.16)	1.55	144.86	0.30 (0.98)	0
33	Barbara	1953/08/15	0.07 (0.23)	1.77	151.48	0.38 (1.25)	0
34	Hazel	1954/10/16	1.56 (5.12)	8.82	152.15	2.35 (7.71)	21
35	Connie	1955/08/13	0.42 (1.38)	4.24	153.97	1.04 (3.41)	12
36	Diane	1955/08/19	0.34 (1.12)	5.04	147.82	0.82 (2.69)	27
37	Ione	1955/09/18	0.01 (0.03)	1.49	13.01	0.29 (0.95)	0
38	Brenda	1960/07/30	0.30 (0.98)	3.38	145.69	0.47 (1.54)	18
39	Donna	1960/09/13	0.15 (0.49)	2.49	155.55	0.70 (2.30)	0
40	Doria	1967/09/11	0.02 (0.07)	1.20	153.00	0.59 (1.94)	0
41	Doria	1971/08/28	0.24 (0.79)	3.10	153.85	0.72 (2.36)	9
42	Bret	1981/07/01	0.02 (0.07)	1.11	144.03	0.54 (1.77)	0
43	Dean	1983/09/28	0.01 (0.03)	0.54	31.95	0.33 (1.08)	0
44	Gloria	1985/09/25	0.01 (0.03)	1.76	12.00	0.29 (0.95)	0
45	Charley	1986/08/16	0.01 (0.03)	0.48	31.85	0.25 (0.82)	0
46	Danielle	1992/09/26	0.07 (0.23)	1.70	149.00	0.34 (1.12)	0
47	Bertha	1996/07/13	0.21 (0.69)	3.29	153.00	0.79 (2.59)	6
48	Fran	1996/09/07	0.42 (1.38)	4.00	149.47	1.25 (4.10)	27
49	Bonnie	1998/08/29	0.08 (0.26)	1.88	150.66	0.36 (1.18)	0
50	Earl	1998/09/05	0.23 (0.75)	3.02	154.70	0.55 (1.80)	15
51	Floyd	1999/09/16	0.23 (0.75)	3.40	155.00	0.88 (2.89)	9
52	Isabel	2003/09/19	0.71 (2.33)	5.92	154.38	2.16 (7.09)	18

[1]Storm duration is the time during a storm when H_S > 0.15 m.

Table A8
Maximum H_s by Storm, Poplar Island, Station 33, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/23	1.27 (4.17)	7.00	321.13	0.17 (0.56)	48
2	1956/10/17	0.57 (1.87)	4.22	328.89	0.58 (1.90)	39
3	1956/10/28	0.77 (2.53)	5.39	325.00	0.65 (2.13)	108
4	1957/10/06	0.82 (2.69)	5.58	324.00	0.56 (1.84)	45
5	1958/02/17	0.96 (3.15)	3.68	331.97	0.24 (0.79)	123
6	1958/10/21	0.58 (1.90)	4.71	325.00	0.39 (1.28)	57
7	1962/03/08	0.86 (2.82)	5.74	323.12	0.38 (1.25)	63
8	1962/11/27	0.63 (2.07)	4.86	325.00	0.29 (0.95)	225
9	1966/01/31	0.90 (2.95)	3.69	276.95	0.05 (0.16)	108
10	1969/01/22	0.42 (1.38)	3.61	333.00	0.45 (1.48)	60
11	1972/05/26	0.57 (1.87)	4.25	328.00	0.58 (1.90)	72
12	1972/10/08	0.85 (2.79)	3.50	330.00	0.32 (1.05)	60
13	1974/12/04	0.88 (2.89)	3.50	332.96	0.48 (1.57)	72
14	1975/07/01	0.78 (2.56)	5.40	323.12	0.11 (0.36)	60
15	1977/10/30	0.54 (1.77)	4.20	327.00	0.42 (1.38)	84
16	1978/04/28	0.65 (2.13)	4.90	325.23	0.56 (1.84)	48
17	1980/12/30	0.56 (1.84)	4.30	325.23	0.41 (1.35)	93
18	1981/08/21	0.57 (1.87)	4.20	328.00	0.64 (2.10)	30
19	1983/02/12	0.58 (1.90)	4.30	328.89	0.57 (1.87)	87
20	1984/03/30	1.17 (3.84)	3.90	330.01	0.94 (3.08)	60
21	1984/09/30	0.60 (1.97)	4.30	328.00	0.76 (2.49)	135
22	1984/10/14	0.71 (2.33)	5.11	325.00	0.51 (1.67)	69
23	1984/11/21	0.54 (1.77)	4.36	324.24	0.07 (0.23)	75
24	1985/10/29	0.48 (1.57)	3.93	329.21	0.29 (0.95)	111
25	1986/12/01	0.56 (1.84)	4.27	327.00	0.38 (1.25)	57
26	1987/02/18	0.47 (1.54)	3.91	330.11	0.25 (0.82)	27
27	1988/04/14	0.55 (1.80)	4.18	328.00	0.67 (2.20)	54
28	1989/03/10	0.52 (1.71)	4.04	327.11	0.49 (1.61)	96
29	1991/01/09	0.41 (1.35)	3.55	333.00	0.51 (1.67)	57
30	1991/04/21	0.37 (1.21)	3.31	331.20	0.45 (1.48)	9
31	1991/10/31	0.44 (1.44)	3.76	329.21	0.35 (1.15)	45
32	1991/11/10	0.60 (1.97)	4.36	328.00	0.57 (1.87)	51
33	1993/03/15	0.90 (2.95)	3.52	277.04	0.69 (2.26)	51
34	1994/10/16	0.47 (1.54)	4.03	326.00	0.23 (0.75)	39
35	1996/10/09	0.57 (1.87)	4.21	327.11	0.58 (1.90)	84
36	1997/06/04	0.42 (1.38)	3.61	333.00	0.66 (2.17)	48
37	1997/10/16	0.56 (1.84)	4.25	326.12	0.38 (1.25)	117
38	1998/05/13	0.41 (1.35)	3.67	331.00	0.48 (1.57)	45
39	1999/05/03	0.46 (1.51)	3.90	331.00	0.53 (1.74)	42
40	1999/08/31	0.55 (1.80)	4.64	323.12	0.12 (0.39)	114
41	2000/05/30	0.52 (1.71)	4.24	326.00	0.24 (0.79)	33
42	2003/04/11	0.49 (1.61)	4.02	327.88	0.54 (1.77)	72
43	2003/09/10	0.42 (1.38)	3.68	331.00	0.32 (1.05)	15

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table A9
Maximum H_S by Storm, Poplar Island, Station 34, Extratropical Storms

Storm Number	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/23	1.05 (3.44)	7.00	317.14	0.17 (0.56)	48
2	1956/10/17	0.54 (1.77)	4.22	326.88	0.58 (1.90)	39
3	1956/10/28	0.66 (2.17)	5.39	322.00	0.65 (2.13)	96
4	1957/10/06	0.69 (2.26)	5.58	321.00	0.56 (1.84)	45
5	1958/02/17	0.99 (3.25)	3.68	330.96	0.24 (0.79)	117
6	1958/10/21	0.52 (1.71)	4.26	324.88	0.54 (1.77)	54
7	1962/03/08	0.73 (2.39)	5.74	320.13	0.38 (1.25)	63
8	1962/11/29	0.57 (1.87)	4.26	327.88	0.84 (2.76)	225
9	1966/01/31	0.92 (3.02)	3.69	277.95	0.05 (0.16)	117
10	1969/01/22	0.40 (1.31)	3.61	330.00	0.45 (1.48)	54
11	1972/05/26	0.54 (1.77)	4.25	326.00	0.58 (1.90)	66
12	1972/10/08	0.88 (2.89)	3.50	329.00	0.32 (1.05)	51
13	1974/12/04	0.90 (2.95)	3.50	331.95	0.48 (1.57)	72
14	1975/07/01	0.65 (2.13)	5.40	319.14	0.11 (0.36)	60
15	1977/10/30	0.50 (1.64)	4.20	324.00	0.42 (1.38)	78
16	1978/04/28	0.58 (1.90)	4.90	322.25	0.56 (1.84)	48
17	1980/12/30	0.52 (1.71)	4.30	322.25	0.41 (1.35)	93
18	1981/08/21	0.54 (1.77)	4.20	326.00	0.64 (2.10)	30
19	1983/02/12	0.55 (1.80)	4.30	326.88	0.57 (1.87)	81
20	1984/03/30	1.18 (3.87)	3.90	329.01	0.94 (3.08)	60
21	1984/09/30	0.56 (1.84)	4.30	326.00	0.76 (2.49)	135
22	1984/10/14	0.61 (2.00)	5.11	322.00	0.51 (1.67)	66
23	1984/11/20	0.50 (1.64)	4.21	323.12	0.30 (0.98)	75
24	1985/10/29	0.45 (1.48)	3.93	326.23	0.29 (0.95)	108
25	1986/12/01	0.52 (1.71)	4.27	324.00	0.38 (1.25)	54
26	1987/02/18	0.44 (1.44)	3.91	327.11	0.25 (0.82)	24
27	1988/04/14	0.53 (1.74)	4.18	326.00	0.67 (2.20)	42
28	1989/03/10	0.50 (1.64)	4.04	325.12	0.49 (1.61)	93
29	1991/01/09	0.39 (1.28)	3.55	330.00	0.51 (1.67)	57
30	1991/04/21	0.35 (1.15)	3.31	328.22	0.45 (1.48)	9
31	1991/10/31	0.41 (1.35)	3.76	326.23	0.35 (1.15)	45
32	1991/11/10	0.57 (1.87)	4.36	326.00	0.57 (1.87)	48
33	1993/03/15	0.85 (2.79)	3.52	277.04	0.69 (2.26)	51
34	1994/10/15	0.44 (1.44)	3.88	328.00	0.26 (0.85)	39
35	1996/10/09	0.54 (1.77)	4.21	325.12	0.58 (1.90)	72
36	1997/06/04	0.40 (1.31)	3.61	330.00	0.66 (2.17)	48
37	1997/10/16	0.52 (1.71)	4.25	323.12	0.38 (1.25)	96
38	1998/05/13	0.39 (1.28)	3.67	328.00	0.48 (1.57)	45
39	1999/05/03	0.44 (1.44)	3.90	328.00	0.53 (1.74)	39
40	1999/09/01	0.52 (1.71)	4.13	326.00	0.56 (1.84)	111
41	2000/05/30	0.47 (1.54)	4.24	322.00	0.24 (0.79)	33
42	2003/04/12	0.46 (1.51)	3.91	329.11	0.56 (1.84)	66
43	2003/09/10	0.39 (1.28)	3.68	328.00	0.32 (1.05)	9

[1]Storm duration is the time during a storm when H_S > 0.3 m.

Table A10
Maximum H_s by Storm, Poplar Island, Station 35, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/23	0.94 (3.08)	7.00	320.13	0.19 (0.62)	45
2	1956/10/17	0.50 (1.64)	4.22	327.88	0.58 (1.90)	30
3	1956/10/28	0.60 (1.97)	5.39	323.00	0.66 (2.17)	108
4	1957/10/06	0.61 (2.00)	5.58	322.00	0.57 (1.87)	39
5	1958/02/17	1.00 (3.28)	3.68	331.97	0.25 (0.82)	54
6	1958/10/22	0.62 (2.03)	2.86	45.01	1.05 (3.44)	63
7	1962/03/06	0.77 (2.53)	3.19	41.00	0.24 (0.79)	72
8	1962/11/29	0.54 (1.77)	4.26	328.89	0.83 (2.72)	213
9	1966/01/28	0.87 (2.85)	3.46	331.97	0.10 (0.33)	69
10	1969/01/21	0.44 (1.44)	2.47	41.00	0.42 (1.38)	48
11	1972/05/26	0.51 (1.67)	4.25	327.00	0.58 (1.90)	60
12	1972/10/08	0.88 (2.89)	3.50	330.00	0.33 (1.08)	66
13	1974/12/04	0.91 (2.99)	3.50	332.96	0.49 (1.61)	42
14	1975/07/01	0.58 (1.90)	5.40	321.13	0.12 (0.39)	54
15	1977/10/30	0.46 (1.51)	4.20	326.00	0.43 (1.41)	72
16	1978/04/28	0.54 (1.77)	4.90	323.24	0.57 (1.87)	51
17	1980/12/29	0.50 (1.64)	2.60	42.02	0.55 (1.80)	93
18	1981/08/21	0.51 (1.67)	4.20	327.00	0.64 (2.10)	48
19	1983/02/12	0.83 (2.72)	3.30	43.00	0.51 (1.67)	93
20	1984/03/30	1.19 (3.90)	3.90	330.01	0.94 (3.08)	54
21	1984/09/30	0.53 (1.74)	4.30	327.00	0.76 (2.49)	126
22	1984/10/14	0.55 (1.80)	5.11	324.00	0.52 (1.71)	63
23	1984/11/20	0.46 (1.51)	4.21	325.12	0.31 (1.02)	63
24	1985/11/05	0.73 (2.39)	3.11	47.98	1.11 (3.64)	117
25	1986/12/01	0.73 (2.39)	3.12	43.00	0.61 (2.00)	42
26	1987/02/18	0.41 (1.35)	3.91	327.11	0.26 (0.85)	15
27	1988/04/14	0.49 (1.61)	4.18	327.00	0.67 (2.20)	36
28	1989/03/10	0.46 (1.51)	4.04	326.12	0.50 (1.64)	93
29	1991/01/09	0.36 (1.18)	3.55	330.00	0.51 (1.67)	45
30	1991/04/20	0.33 (1.08)	2.14	42.02	0.61 (2.00)	9
31	1991/10/31	0.38 (1.25)	3.76	326.23	0.36 (1.18)	24
32	1991/11/10	0.53 (1.74)	4.36	327.00	0.57 (1.87)	45
33	1993/03/14	0.49 (1.61)	4.14	325.23	0.74 (2.43)	21
34	1994/10/15	0.40 (1.31)	3.88	328.00	0.27 (0.89)	27
35	1996/10/09	0.51 (1.67)	2.65	41.00	0.44 (1.44)	69
36	1997/06/04	0.38 (1.25)	3.61	330.00	0.67 (2.20)	42
37	1997/10/16	0.48 (1.57)	4.25	325.12	0.38 (1.25)	75
38	1998/05/13	0.36 (1.18)	3.67	328.00	0.48 (1.57)	27
39	1999/05/03	0.40 (1.31)	3.90	328.00	0.54 (1.77)	27
40	1999/09/01	0.48 (1.57)	4.13	327.00	0.56 (1.84)	102
41	2000/05/30	0.43 (1.41)	4.24	324.00	0.24 (0.79)	27
42	2003/04/12	0.44 (1.44)	3.91	329.11	0.57 (1.87)	48
43	2003/09/12	0.54 (1.77)	2.72	40.02	0.53 (1.74)	9

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table A11
Maximum H_S by Storm, Poplar Island, Station 36, Extratropical Storms

Storm Number	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/23	0.90 (2.95)	7.00	321.13	0.19 (0.62)	45
2	1956/10/17	0.49 (1.61)	4.22	327.88	0.58 (1.90)	33
3	1956/10/28	0.58 (1.90)	5.39	323.00	0.66 (2.17)	108
4	1957/10/06	0.59 (1.94)	5.58	323.00	0.57 (1.87)	39
5	1958/02/17	0.99 (3.25)	3.68	331.97	0.25 (0.82)	45
6	1958/10/22	0.63 (2.07)	2.86	45.01	1.05 (3.44)	60
7	1962/03/06	0.79 (2.59)	3.19	42.00	0.24 (0.79)	72
8	1962/11/29	0.52 (1.71)	4.26	328.89	0.83 (2.72)	201
9	1966/01/28	0.82 (2.69)	3.46	331.97	0.10 (0.33)	63
10	1969/01/21	0.45 (1.48)	2.47	42.00	0.42 (1.38)	42
11	1972/05/26	0.50 (1.64)	4.25	327.00	0.58 (1.90)	51
12	1972/10/08	0.88 (2.89)	3.50	330.00	0.33 (1.08)	63
13	1974/12/04	0.90 (2.95)	3.50	332.96	0.49 (1.61)	42
14	1975/07/01	0.56 (1.84)	5.40	321.13	0.12 (0.39)	48
15	1977/10/30	0.45 (1.48)	4.20	326.00	0.43 (1.41)	57
16	1978/04/28	0.52 (1.71)	4.90	323.24	0.57 (1.87)	48
17	1980/12/29	0.51 (1.67)	2.60	43.01	0.55 (1.80)	87
18	1981/08/21	0.50 (1.64)	4.20	327.00	0.64 (2.10)	48
19	1983/02/12	0.85 (2.79)	3.30	44.00	0.51 (1.67)	87
20	1984/03/30	1.17 (3.84)	3.90	330.01	0.94 (3.08)	51
21	1984/09/30	0.51 (1.67)	4.30	327.00	0.76 (2.49)	114
22	1984/10/14	0.54 (1.77)	5.11	324.00	0.52 (1.71)	57
23	1984/11/20	0.45 (1.48)	4.21	325.12	0.31 (1.02)	60
24	1985/11/05	0.75 (2.46)	3.11	47.98	1.11 (3.64)	105
25	1986/12/03	0.75 (2.46)	3.12	44.00	0.61 (2.00)	42
26	1987/02/18	0.39 (1.28)	3.91	327.11	0.26 (0.85)	15
27	1988/04/14	0.48 (1.57)	4.18	327.00	0.67 (2.20)	33
28	1989/03/10	0.45 (1.48)	4.04	326.12	0.50 (1.64)	84
29	1991/01/09	0.36 (1.18)	3.55	329.00	0.51 (1.67)	24
30	1991/04/20	0.34 (1.12)	2.14	43.01	0.61 (2.00)	9
31	1991/10/31	0.36 (1.18)	3.76	326.23	0.36 (1.18)	18
32	1991/11/10	0.52 (1.71)	4.36	327.00	0.57 (1.87)	36
33	1993/03/14	0.47 (1.54)	4.14	325.23	0.74 (2.43)	18
34	1994/10/15	0.39 (1.28)	3.88	328.00	0.27 (0.89)	21
35	1996/10/09	0.52 (1.71)	2.65	42.00	0.44 (1.44)	69
36	1997/06/04	0.37 (1.21)	3.61	329.00	0.67 (2.20)	39
37	1997/10/16	0.46 (1.51)	4.25	325.12	0.38 (1.25)	51
38	1998/05/13	0.35 (1.15)	3.67	328.00	0.48 (1.57)	21
39	1999/05/03	0.39 (1.28)	3.90	328.00	0.54 (1.77)	24
40	1999/09/01	0.47 (1.54)	4.13	327.00	0.56 (1.84)	102
41	2000/05/30	0.42 (1.38)	4.24	325.00	0.24 (0.79)	27
42	2003/04/12	0.43 (1.41)	3.91	328.11	0.57 (1.87)	45
43	2003/09/12	0.55 (1.80)	2.72	41.02	0.53 (1.74)	9

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table A12
Maximum H_s by Storm, Poplar Island, Station 37, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/22	0.01 (0.03)	1.96	337.00	0.19 (0.62)	0
2	1956/10/17	0.23 (0.75)	2.07	45.01	0.48 (1.57)	12
3	1956/10/27	0.47 (1.54)	2.75	48.00	0.95 (3.12)	15
4	1957/10/02	0.01 (0.03)	2.25	335.18	0.29 (0.95)	0
5	1958/02/16	0.01 (0.03)	2.10	339.73	0.19 (0.62)	0
6	1958/10/22	0.52 (1.71)	2.86	48.00	1.05 (3.44)	9
7	1962/03/06	0.59 (1.94)	3.19	46.00	0.24 (0.79)	9
8	1962/11/26	0.01 (0.03)	3.27	337.00	0.29 (0.95)	0
9	1966/01/30	0.31 (1.02)	2.40	48.97	-0.05 (-0.16)	3
10	1969/01/21	0.34 (1.12)	2.47	46.00	0.42 (1.38)	3
11	1972/05/24	0.01 (0.03)	3.37	339.73	0.29 (0.95)	0
12	1972/10/07	0.49 (1.61)	2.90	45.01	0.53 (1.74)	36
13	1974/12/02	0.70 (2.30)	3.40	45.99	1.00 (3.28)	12
14	1975/06/29	0.20 (0.66)	1.95	47.98	0.25 (0.82)	3
15	1977/10/29	0.01 (0.03)	2.60	337.00	0.29 (0.95)	0
16	1978/04/26	0.27 (0.89)	2.21	46.00	0.28 (0.92)	9
17	1980/12/29	0.41 (1.35)	2.60	46.01	0.55 (1.80)	12
18	1981/08/21	0.34 (1.12)	2.40	46.01	0.75 (2.46)	30
19	1983/02/12	0.65 (2.13)	3.30	47.00	0.51 (1.67)	24
20	1984/03/29	0.58 (1.90)	3.20	46.99	0.23 (0.75)	9
21	1984/09/27	0.01 (0.03)	3.26	199.33	0.29 (0.95)	0
22	1984/10/11	0.01 (0.03)	2.67	338.82	0.29 (0.95)	0
23	1984/11/19	0.15 (0.49)	1.67	46.00	0.29 (0.95)	0
24	1985/11/05	0.61 (2.00)	3.11	51.00	1.11 (3.64)	30
25	1986/12/03	0.58 (1.90)	3.12	47.00	0.61 (2.00)	9
26	1987/02/15	0.01 (0.03)	2.37	337.00	0.19 (0.62)	0
27	1988/04/12	0.23 (0.75)	2.07	45.01	0.14 (0.46)	3
28	1989/03/07	0.01 (0.03)	2.99	337.00	0.19 (0.62)	0
29	1991/01/07	0.01 (0.03)	1.86	336.09	0.19 (0.62)	0
30	1991/04/20	0.27 (0.89)	2.14	46.01	0.61 (2.00)	9
31	1991/10/28	0.01 (0.03)	1.36	337.91	0.29 (0.95)	0
32	1991/11/08	0.01 (0.03)	1.48	334.27	0.29 (0.95)	0
33	1993/03/13	0.01 (0.03)	1.61	288.34	0.19 (0.62)	0
34	1994/10/13	0.25 (0.82)	2.07	47.00	0.62 (2.03)	6
35	1996/10/09	0.39 (1.28)	2.65	46.00	0.44 (1.44)	21
36	1997/06/02	0.27 (0.89)	2.13	46.01	0.63 (2.07)	18
37	1997/10/14	0.01 (0.03)	2.98	192.83	0.29 (0.95)	0
38	1998/05/14	0.14 (0.46)	1.63	46.00	0.54 (1.77)	0
39	1999/05/02	0.20 (0.66)	1.90	46.99	0.46 (1.51)	6
40	1999/09/05	0.32 (1.05)	2.40	46.00	0.35 (1.15)	9
41	2000/05/29	0.17 (0.56)	1.78	47.98	0.29 (0.95)	9
42	2003/04/08	0.01 (0.03)	3.39	337.00	0.29 (0.95)	0
43	2003/09/12	0.42 (1.38)	2.72	45.01	0.53 (1.74)	3

[1]Storm duration is the time during a storm when H_S > 0.15 m.

Table A13
Maximum H_S by Storm, Poplar Island, Station 38, Extratropical Storms

Storm Number	Date	H_S, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/22	0.01 (0.03)	1.96	346.03	0.19 (0.62)	0
2	1956/10/17	0.18 (0.59)	2.07	36.03	0.48 (1.57)	6
3	1956/10/27	0.41 (1.35)	2.75	47.00	0.96 (3.15)	15
4	1957/10/02	0.01 (0.03)	2.25	355.07	0.29 (0.95)	0
5	1958/02/16	0.09 (0.30)	3.21	155.04	0.19 (0.62)	0
6	1958/10/22	0.45 (1.48)	2.86	47.00	1.06 (3.48)	9
7	1962/03/06	0.46 (1.51)	3.19	37.00	0.25 (0.82)	9
8	1962/11/26	0.01 (0.03)	3.27	357.00	0.29 (0.95)	0
9	1966/01/30	0.24 (0.79)	2.40	35.88	-0.04 (-0.13)	3
10	1969/01/21	0.26 (0.85)	2.47	37.00	0.43 (1.41)	3
11	1972/05/25	0.11 (0.36)	3.25	136.89	0.15 (0.49)	0
12	1972/10/07	0.40 (1.31)	2.70	49.00	0.87 (2.85)	36
13	1974/12/02	0.56 (1.84)	3.40	40.98	1.00 (3.28)	24
14	1975/06/29	0.16 (0.52)	1.95	34.92	0.25 (0.82)	3
15	1977/10/29	0.01 (0.03)	2.60	357.00	0.29 (0.95)	0
16	1978/04/26	0.21 (0.69)	2.21	37.00	0.28 (0.92)	9
17	1980/12/29	0.33 (1.08)	2.60	41.02	0.55 (1.80)	12
18	1981/08/21	0.28 (0.92)	2.40	41.02	0.75 (2.46)	30
19	1983/02/12	0.54 (1.77)	3.30	42.00	0.51 (1.67)	24
20	1984/03/29	0.45 (1.48)	3.20	37.97	0.24 (0.79)	12
21	1984/09/27	0.13 (0.43)	3.26	150.27	0.29 (0.95)	0
22	1984/10/11	0.01 (0.03)	2.67	358.93	0.29 (0.95)	0
23	1984/11/19	0.11 (0.36)	1.67	37.00	0.29 (0.95)	0
24	1985/11/05	0.54 (1.77)	3.11	49.99	1.12 (3.67)	39
25	1986/12/03	0.48 (1.57)	3.12	42.00	0.62 (2.03)	9
26	1987/02/15	0.01 (0.03)	2.37	357.00	0.19 (0.62)	0
27	1988/04/12	0.18 (0.59)	2.07	32.04	0.15 (0.49)	3
28	1989/03/07	0.01 (0.03)	2.99	357.00	0.19 (0.62)	0
29	1991/01/07	0.01 (0.03)	1.86	356.04	0.19 (0.62)	0
30	1991/04/20	0.22 (0.72)	2.14	41.02	0.61 (2.00)	9
31	1991/10/28	0.01 (0.03)	1.36	357.96	0.29 (0.95)	0
32	1991/11/08	0.01 (0.03)	1.48	354.11	0.29 (0.95)	0
33	1993/03/13	0.06 (0.20)	2.23	144.55	0.19 (0.62)	0
34	1994/10/13	0.21 (0.69)	2.07	42.00	0.62 (2.03)	6
35	1996/10/09	0.30 (0.98)	2.65	37.00	0.44 (1.44)	21
36	1997/06/02	0.22 (0.72)	2.13	41.02	0.63 (2.07)	15
37	1997/10/14	0.11 (0.36)	2.98	145.37	0.29 (0.95)	0
38	1998/05/14	0.11 (0.36)	1.63	37.00	0.55 (1.80)	0
39	1999/05/02	0.15 (0.49)	1.90	37.97	0.46 (1.51)	0
40	1999/09/05	0.26 (0.85)	2.34	44.94	0.56 (1.84)	9
41	2000/05/29	0.13 (0.43)	1.78	38.94	0.29 (0.95)	0
42	2003/04/08	0.01 (0.03)	3.39	357.00	0.29 (0.95)	0
43	2003/09/12	0.32 (1.05)	2.72	36.03	0.53 (1.74)	3

[1]Storm duration is the time during a storm when H_S > 0.15 m.

Table A14
Maximum H_s by Storm, Poplar Island, Station 39, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1954/01/22	0.01 (0.03)	1.96	359.07	0.19 (0.62)	0
2	1956/10/16	0.01 (0.03)	1.69	12.00	0.29 (0.95)	0
3	1956/10/24	0.01 (0.03)	2.58	10.00	0.29 (0.95)	0
4	1957/10/02	0.01 (0.03)	2.25	8.99	0.29 (0.95)	0
5	1958/02/16	0.19 (0.62)	3.21	152.00	0.20 (0.66)	6
6	1958/10/20	0.01 (0.03)	2.90	12.00	0.29 (0.95)	0
7	1962/03/05	0.01 (0.03)	2.74	10.00	0.19 (0.62)	0
8	1962/11/26	0.01 (0.03)	3.27	11.00	0.29 (0.95)	0
9	1966/01/31	0.23 (0.75)	3.80	153.14	0.31 (1.02)	15
10	1969/01/20	0.01 (0.03)	1.61	0.14	0.19 (0.62)	0
11	1972/05/25	0.25 (0.82)	3.25	141.78	0.16 (0.52)	6
12	1972/10/04	0.01 (0.03)	2.98	13.01	0.29 (0.95)	0
13	1974/12/02	0.40 (1.31)	5.10	150.00	0.92 (3.02)	33
14	1975/06/29	0.17 (0.56)	2.63	144.03	0.29 (0.95)	3
15	1977/10/29	0.01 (0.03)	2.60	11.00	0.29 (0.95)	0
16	1978/04/26	0.01 (0.03)	1.94	31.00	0.29 (0.95)	0
17	1980/12/27	0.01 (0.03)	2.80	10.00	0.19 (0.62)	0
18	1981/08/20	0.24 (0.79)	3.10	147.34	0.41 (1.35)	9
19	1983/02/11	0.01 (0.03)	3.20	10.00	0.19 (0.62)	0
20	1984/03/30	0.35 (1.15)	3.95	159.31	0.84 (2.76)	9
21	1984/09/27	0.27 (0.89)	3.26	152.31	0.29 (0.95)	3
22	1984/10/11	0.01 (0.03)	2.67	13.01	0.29 (0.95)	0
23	1984/11/19	0.01 (0.03)	1.67	31.00	0.29 (0.95)	0
24	1985/11/06	0.29 (0.95)	3.31	155.86	1.06 (3.48)	21
25	1986/12/03	0.14 (0.46)	2.40	152.42	1.01 (3.31)	0
26	1987/02/15	0.01 (0.03)	2.37	11.00	0.19 (0.62)	0
27	1988/04/12	0.01 (0.03)	2.28	8.99	0.29 (0.95)	0
28	1989/03/07	0.01 (0.03)	2.99	11.00	0.19 (0.62)	0
29	1991/01/07	0.01 (0.03)	1.86	10.00	0.19 (0.62)	0
30	1991/04/20	0.11 (0.36)	2.48	153.00	0.67 (2.20)	0
31	1991/10/28	0.01 (0.03)	1.36	12.00	0.29 (0.95)	0
32	1991/11/08	0.01 (0.03)	1.48	7.99	0.29 (0.95)	0
33	1993/03/13	0.12 (0.39)	2.23	146.52	0.19 (0.62)	0
34	1994/10/13	0.08 (0.26)	2.12	149.00	0.52 (1.71)	0
35	1996/10/09	0.15 (0.49)	2.46	150.66	0.37 (1.21)	0
36	1997/06/06	0.23 (0.75)	3.00	153.00	0.66 (2.17)	6
37	1997/10/14	0.22 (0.72)	2.98	147.34	0.29 (0.95)	9
38	1998/05/14	0.04 (0.13)	1.48	154.02	0.68 (2.23)	0
39	1999/04/29	0.15 (0.49)	2.63	145.00	0.16 (0.52)	0
40	1999/09/06	0.21 (0.69)	2.87	150.45	0.78 (2.56)	18
41	2000/05/31	0.03 (0.10)	1.21	153.00	0.77 (2.53)	0
42	2003/04/08	0.01 (0.03)	3.39	11.00	0.29 (0.95)	0
43	2003/09/08	0.01 (0.03)	1.50	10.00	0.29 (0.95)	0

[1]Storm duration is the time during a storm when H_s > 0.15 m.

Appendix B
Extremal Wave and Water Level Analysis Results for Poplar Island

Extremal analysis of significant wave heights was applied to all storms together and to hurricanes only. The results are summarized in this appendix. Analysis of all storms included 179 storms over the 148-year time period. Analysis of hurricanes only included 52 storms over the 148-year period. The best-fitting extremal distribution was selected, based on the criteria of Goda and Kobune (1990) and a good visual fit to the return periods of concern for this project. Using the best-fit distribution, significant wave heights were determined for return periods of 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, and 100 years. For hurricane-influenced stations where the best-fit distribution for all storms underestimated H_s at the longest return periods, return period H_s was taken from the best-fit for hurricanes only for return periods dominated by hurricanes.

To estimate an appropriate peak wave period and water level to accompany each return-period significant wave height, the computer program return_period_Tp.f is run. Inputs include return-period significant wave heights and 148-year time history of waves and water levels at each station. The time history is screened to find all significant heights within a bin centered on the desired return period wave height. Bin widths considered are 0.2, 0.4, 0.6, 0.8, and 1.0 m (0.7, 1.3, 2.0, 2.6, 3.3 ft). For example, the 50-year significant height at Poplar Island sta 2 is 2.24 m (7.34 ft). All cases in the 148-year sta 2 time history with significant height in the range 2.14-2.34 m (0.2-m bin) [7.02-7.68 ft (0.7-ft bin)] were identified, and their peak periods and water levels were averaged. The process was repeated for significant heights in the range 2.04-2.44 m (0.4-m bin) [6.69-8.01 ft (1.3-ft bin)], 1.94-2.54 m (0.6-m bins) [6.36-8.33 ft (2.0-ft bin)], 1.84-2.64 m (0.8-m bins) [6.04-8.66 ft (2.6-ft bin)], and 1.74-2.74 m (1.0-m bin) [5.71-8.99 ft (3.3-ft bin)]. For each return period, a representative or *average* period and water level were chosen with consideration of bins that captured enough cases to form a meaningful average but not so many cases as to dilute the target severe events.

Tables B1-B16 summarize results from the extremal analysis of waves for sta 1-16. Tables B17-B23 give extremal results for sta 33-39. The extremal values are plotted as a function of return period in Figures B1-B7 for sta 33-39. In Figures B1-B7, average water levels associated with extremal wave heights have been summed with depths relative to mllw to get overall average depths for each return period and each station.

The last two tables, Tables B24 and B25, give results of an extremal analysis of maximum storm water levels for Poplar Island. The maximum water levels for each storm were fit to a Fisher-Tippett type I distribution. The extremal water levels, referenced to msl, associated with northeasters from Table 16 in Chapter 3 are listed in Table B24. The extremal water levels, referenced to msl, associated with tropical storms from Table 15 in Chapter 3 are listed in Table B25. The relationship used here for Poplar Island tidal datums is msl = 0.230 m mllw. It should be noted that these extremal water levels in Tables B24 and B25 are only for storms. The extremal analysis did not include all water levels throughout the year (e.g., spring tide).

Table B1
Poplar Island Station 1 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.77 (2.53)	3.32	0.70 (2.30)
10	1.11 (3.64)	4.44	1.06 (3.48)
15	1.33 (4.36)	4.68	1.12 (3.67)
20	1.49 (4.89)	5.16	1.48 (4.86)
25	1.62 (5.31)	5.60	1.75 (5.74)
30	1.72 (5.64)	5.88	1.77 (5.81)
35	1.82 (5.97)	6.02	1.75 (5.74)
40	1.89 (6.20)	6.58	1.72 (5.64)
45	1.97 (6.46)	6.55	1.88 (6.17)
50	2.03 (6.66)	6.79	2.01 (6.59)
100	2.47 (8.10)	8.64	2.61 (8.56)

Table B2 Poplar Island Station 2 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.79 (2.59)	3.31	0.70 (2.30)
10	1.18 (3.87)	4.38	1.03 (3.38)
15	1.43 (4.69)	4.77	1.08 (3.54)
20	1.62 (5.31)	5.29	1.52 (4.99)
25	1.77 (5.81)	5.52	1.77 (5.81)
30	1.89 (6.20)	5.63	1.81 (5.94)
35	2.00 (6.56)	6.26	1.73 (5.68)
40	2.09 (6.86)	6.55	1.84 (6.04)
45	2.17 (7.12)	6.74	1.94 (6.36)
50	2.24 (7.35)	6.79	2.01 (6.59)
100	2.58 (8.46)	8.64	2.61 (8.56)

Table B3 Poplar Island Station 3 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.95 (3.12)	3.79	0.91 (2.99)
10	1.41 (4.63)	4.52	1.06 (3.48)
15	1.70 (5.58)	5.00	1.20 (3.94)
20	1.92 (6.30)	5.42	1.13 (3.71)
25	2.09 (6.86)	5.71	1.08 (3.54)
30	2.23 (7.32)	5.60	1.60 (5.25)
35	2.36 (7.74)	5.86	1.80 (5.91)
40	2.46 (8.07)	5.93	1.78 (5.84)
45	2.56 (8.40)	5.98	1.84 (6.04)
50	2.64 (8.66)	6.12	1.80 (5.91)
100	3.23 (10.60)	7.94	2.44 (8.01)

Table B4 Poplar Island Station 4 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	1.13 (3.71)	4.05	0.93 (3.05)
10	1.61 (5.28)	4.97	1.18 (3.87)
15	1.91 (6.27)	5.23	1.26 (4.13)
20	2.13 (6.99)	5.53	1.18 (3.87)
25	2.31 (7.58)	5.75	1.34 (4.40)
30	2.54 (8.33)	5.98	1.78 (5.84)
35	2.69 (8.83)	6.21	1.55 (5.09)
40	2.83 (9.28)	6.58	1.48 (4.86)
45	2.96 (9.71)	6.69	1.27 (4.17)
50	3.07 (10.07)	6.53	1.75 (5.74)
100	3.78 (12.40)	7.60	1.67 (5.48)

Table B5 Poplar Island Station 5 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	1.14 (3.74)	4.99	0.57 (1.87)
10	1.46 (4.79)	5.13	1.17 (3.84)
15	1.66 (5.45)	5.48	1.21 (3.97)
20	1.81 (5.94)	5.97	1.46 (4.79)
25	1.93 (6.33)	6.11	1.41 (4.63)
30	2.03 (6.66)	6.01	1.70 (5.58)
35	2.11 (6.92)	6.05	1.70 (5.58)
40	2.20 (7.22)	6.12	1.82 (5.97)
45	2.29 (7.51)	6.22	1.93 (6.33)
50	2.36 (7.74)	6.22	1.93 (6.33)
100	2.86 (9.38)	7.55	1.97 (6.46)

Table B6 **Poplar Island Station 6 Extremal Wave Analysis Results**			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.98 (3.22)	4.64	0.44 (1.44)
10	1.29 (4.23)	5.48	0.89 (2.92)
15	1.50 (4.92)	5.76	1.16 (3.81)
20	1.64 (5.38)	6.03	1.22 (4.00)
25	1.76 (5.77)	6.03	1.57 (5.15)
30	1.86 (6.10)	6.07	1.76 (5.77)
35	1.95 (6.40)	6.12	1.78 (5.84)
40	2.02 (6.63)	6.22	1.89 (6.20)
45	2.09 (6.86)	6.22	1.89 (6.20)
50	2.15 (7.05)	6.71	1.94 (6.36)
100	2.55 (8.37)	7.81	2.00 (6.56)

Table B7 **Poplar Island Station 7 Extremal Wave Analysis Results**			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	1.23 (4.04)	4.98	0.45 (1.48)
10	1.55 (5.09)	5.58	0.72 (2.36)
15	1.76 (5.77)	5.78	0.94 (3.08)
20	1.91 (6.27)	5.76	1.38 (4.53)
25	2.03 (6.66)	5.79	1.53 (5.02)
30	2.13 (6.99)	6.03	1.49 (4.89)
35	2.22 (7.28)	6.35	1.54 (5.05)
40	2.30 (7.55)	6.55	1.64 (5.38)
45	2.36 (7.74)	6.77	1.67 (5.48)
50	2.42 (7.94)	6.77	1.67 (5.48)
100	2.84 (9.32)	7.94	2.11 (6.92)

Table B8
Poplar Island Station 8 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	1.15 (3.77)	4.98	0.45 (1.48)
10	1.45 (4.76)	5.55	0.70 (2.30)
15	1.64 (5.38)	5.78	0.94 (3.08)
20	1.78 (5.84)	5.64	1.39 (4.56)
25	1.89 (6.20)	5.94	1.53 (5.02)
30	1.99 (6.53)	6.28	1.54 (5.05)
35	2.07 (6.79)	6.35	1.54 (5.05)
40	2.14 (7.02)	6.49	1.61 (5.28)
45	2.20 (7.22)	6.77	1.67 (5.48)
50	2.26 (7.41)	6.77	1.67 (5.48)
100	2.64 (8.66)	7.94	2.11 (6.92)

Table B9
Poplar Island Station 9 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	1.23 (4.04)	4.93	0.42 (1.38)
10	1.54 (5.05)	5.55	0.54 (1.77)
15	1.73 (5.68)	5.89	0.97 (3.18)
20	1.87 (6.14)	5.77	1.45 (4.76)
25	1.99 (6.53)	5.99	1.35 (4.43)
30	2.08 (6.82)	6.36	1.48 (4.86)
35	2.16 (7.09)	6.45	1.66 (5.45)
40	2.23 (7.32)	6.77	1.67 (5.48)
45	2.30 (7.55)	6.77	1.67 (5.48)
50	2.35 (7.71)	6.84	1.76 (5.77)
100	2.74 (8.99)	7.94	2.11 (6.92)

Table B10 **Poplar Island Station 10 Extremal Wave Analysis Results**			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	1.12 (3.67)	5.14	0.44 (1.44)
10	1.32 (4.33)	5.81	0.54 (1.77)
15	1.43 (4.69)	5.99	0.63 (2.07)
20	1.51 (4.95)	6.27	0.83 (2.72)
25	1.57 (5.15)	6.42	0.85 (2.79)
30	1.62 (5.31)	6.28	0.87 (2.85)
35	1.66 (5.45)	6.35	1.34 (4.40)
40	1.69 (5.54)	6.35	1.34 (4.40)
45	1.73 (5.68)	6.35	1.34 (4.40)
50	1.79 (5.87)	6.35	1.34 (4.40)
100	2.23 (7.32)	8.46	1.93 (6.33)

Table B11 **Poplar Island Station 11 Extremal Wave Analysis Results**			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	1.14 (3.74)	5.18	0.50 (1.64)
10	1.34 (4.40)	5.79	0.69 (2.26)
15	1.46 (4.79)	6.06	0.83 (2.72)
20	1.53 (5.02)	6.36	0.78 (2.56)
25	1.59 (5.22)	6.38	0.88 (2.89)
30	1.64 (5.38)	6.43	1.06 (3.48)
35	1.68 (5.51)	6.35	1.19 (3.90)
40	1.73 (5.68)	6.26	1.59 (5.22)
45	1.80 (5.91)	6.82	1.77 (5.81)
50	1.87 (6.14)	6.82	1.77 (5.81)
100	2.32 (7.61)	8.46	1.93 (6.33)

Table B12 Poplar Island Station 12 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.91 (2.99)	4.51	0.37 (1.21)
10	1.13 (3.71)	5.45	0.45 (1.48)
15	1.27 (4.17)	5.67	0.45 (1.48)
20	1.37 (4.49)	6.05	0.34 (1.12)
25	1.45 (4.76)	6.14	0.32 (1.05)
30	1.52 (4.99)	6.17	0.21 (0.69)
35	1.58 (5.18)	6.17	0.21 (0.69)
40	1.63 (5.35)	6.44	0.19 (0.62)
45	1.67 (5.48)	6.64	0.40 (1.31)
50	1.71 (5.61)	7.36	0.68 (2.23)
100	1.99 (6.53)	7.65	0.98 (3.22)

Table B13 Poplar Island Station 13 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.83 (2.72)	4.31	0.35 (1.15)
10	1.00 (3.28)	4.84	0.42 (1.38)
15	1.12 (3.67)	5.37	0.51 (1.67)
20	1.20 (3.94)	5.67	0.45 (1.48)
25	1.26 (4.13)	5.75	0.49 (1.61)
30	1.32 (4.33)	5.98	0.47 (1.54)
35	1.37 (4.49)	6.09	0.53 (1.74)
40	1.41 (4.63)	6.16	0.59 (1.94)
45	1.45 (4.76)	7.02	0.62 (2.03)
50	1.48 (4.86)	7.73	1.07 (3.51)
100	1.50 (4.92)	7.65	0.98 (3.22)

Table B14
Poplar Island Station 14 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.75 (2.46)	4.42	0.36 (1.18)
10	0.87 (2.85)	5.21	0.47 (1.54)
15	0.93 (3.05)	5.30	0.54 (1.77)
20	0.98 (3.22)	5.42	0.71 (2.33)
25	1.01 (3.31)	5.39	0.81 (2.66)
30	1.04 (3.41)	5.52	0.95 (3.12)
35	1.07 (3.51)	5.22	1.25 (4.10)
40	1.09 (3.58)	6.32	1.55 (5.09)
45	1.11 (3.64)	6.32	1.55 (5.09)
50	1.13 (3.71)	6.73	0.72 (2.36)
100	1.24 (4.07)	6.73	0.72 (2.36)

Table B15
Poplar Island Station 15 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.64 (2.10)	4.16	0.49 (1.61)
10	0.75 (2.46)	4.40	0.61 (2.00)
15	0.81 (2.66)	4.36	0.66 (2.17)
20	0.86 (2.82)	4.63	0.87 (2.85)
25	0.89 (2.92)	4.75	0.93 (3.05)
30	0.92 (3.02)	4.75	0.93 (3.05)
35	0.95 (3.12)	4.90	0.84 (2.76)
40	0.97 (3.18)	4.97	0.74 (2.43)
45	0.99 (3.25)	5.12	0.73 (2.40)
50	1.00 (3.28)	5.29	0.75 (2.46)
100	1.26 (4.13)	8.64	2.30 (7.55)

Table B16 Poplar Island Station 16 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level** mllw, m (ft)
5	0.53 (1.74)	3.52	0.78 (2.56)
10	0.65 (2.13)	3.84	0.86 (2.82)
15	0.72 (2.36)	4.05	0.95 (3.12)
20	0.77 (2.53)	4.48	1.22 (4.00)
25	0.80 (2.62)	4.54	1.23 (4.04)
30	0.83 (2.72)	4.96	1.30 (4.27)
35	0.86 (2.82)	4.96	1.30 (4.27)
40	0.88 (2.89)	5.24	1.42 (4.66)
45	0.90 (2.95)	5.03	1.24 (4.07)
50	0.92 (3.02)	5.03	1.24 (4.07)
100	1.03 (3.38)	7.60	1.53 (5.02)

Table B17 Poplar Island Station 33 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.90 (2.95)	4.31	0.27 (0.89)
10	1.08 (3.54)	4.98	0.65 (2.13)
15	1.17 (3.84)	5.98	0.98 (3.22)
20	1.25 (4.10)	5.90	0.91 (2.99)
25	1.30 (4.27)	5.99	0.86 (2.82)
30	1.34 (4.40)	5.99	0.86 (2.82)
35	1.38 (4.53)	6.88	0.82 (2.69)
40	1.41 (4.63)	6.94	0.74 (2.43)
45	1.44 (4.72)	7.02	0.86 (2.82)
50	1.47 (4.82)	7.02	0.86 (2.82)
100	1.96 (6.43)	8.28	1.85 (6.07)

Table B18 Poplar Island Station 34 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.88 (2.89)	4.17	0.30 (0.98)
10	1.07 (3.51)	4.35	0.54 (1.77)
15	1.17 (3.84)	5.49	1.01 (3.31)
20	1.24 (4.07)	5.84	1.13 (3.71)
25	1.30 (4.27)	5.62	1.41 (4.63)
30	1.35 (4.43)	5.50	1.51 (4.95)
35	1.38 (4.53)	6.31	1.89 (6.20)
40	1.54 (5.05)	6.60	1.77 (5.81)
45	1.61 (5.28)	6.72	1.66 (5.45)
50	1.68 (5.51)	6.82	1.82 (5.97)
100	2.12 (6.96)	7.88	1.77 (5.81)

Table B19 Poplar Island Station 35 Extremal Wave Analysis Results			
Return Period, Years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.75 (2.46)	3.87	0.24 (0.79)
10	0.88 (2.89)	3.59	0.19 (0.62)
15	0.96 (3.15)	3.86	0.38 (1.25)
20	1.01 (3.31)	4.19	0.37 (1.21)
25	1.06 (3.48)	3.67	0.51 (1.67)
30	1.09 (3.58)	3.69	0.67 (2.20)
35	1.12 (3.67)	3.80	0.94 (3.08)
40	1.14 (3.74)	3.86	0.88 (2.89)
45	1.16 (3.81)	3.86	0.88 (2.89)
50	1.18 (3.87)	3.86	0.88 (2.89)
100	1.20 (3.94)	3.88	0.94 (3.08)

Table B20 Poplar Island Station 36 Extremal Wave Analysis Results			
Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.75 (2.46)	3.57	0.23 (0.75)
10	0.88 (2.89)	3.55	0.24 (0.79)
15	0.96 (3.15)	3.95	0.44 (1.44)
20	1.01 (3.31)	3.62	0.64 (2.10)
25	1.06 (3.48)	3.66	0.62 (2.03)
30	1.09 (3.58)	3.75	0.88 (2.89)
35	1.12 (3.67)	3.75	0.88 (2.89)
40	1.14 (3.74)	3.80	0.94 (3.08)
45	1.17 (3.84)	3.86	0.88 (2.89)
50	1.19 (3.90)	3.86	0.88 (2.89)
100	1.23 (4.04)	3.86	0.88 (2.89)

Table B21 Poplar Island Station 37 Extremal Wave Analysis Results			
Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.49 (1.61)	2.82	0.64 (2.10)
10	0.61 (2.00)	3.13	0.63 (2.07)
15	0.67 (2.20)	3.23	0.61 (2.00)
20	0.71 (2.33)	3.41	0.85 (2.79)
25	0.74 (2.43)	3.49	0.79 (2.59)
30	0.77 (2.53)	3.55	0.89 (2.92)
35	0.79 (2.59)	3.55	0.89 (2.92)
40	0.81 (2.66)	3.65	0.85 (2.79)
45	0.82 (2.69)	3.65	0.85 (2.79)
50	0.83 (2.72)	3.70	0.93 (3.05)
100	0.92 (3.02)	3.80	0.78 (2.56)

Table B22
Poplar Island Station 38 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.43 (1.41)	3.06	0.75 (2.46)
10	0.52 (1.71)	3.11	0.67 (2.20)
15	0.57 (1.87)	3.27	0.82 (2.69)
20	0.61 (2.00)	3.35	0.88 (2.89)
25	0.63 (2.07)	3.41	0.90 (2.95)
30	0.65 (2.13)	3.89	1.05 (3.44)
35	0.67 (2.20)	4.25	1.05 (3.44)
40	0.68 (2.23)	4.25	1.05 (3.44)
45	0.69 (2.26)	5.11	1.34 (4.40)
50	0.70 (2.30)	5.11	1.34 (4.40)
100	0.77 (2.53)	6.22	1.57 (5.15)

Table B23
Poplar Island Station 39 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.33 (1.08)	3.80	0.85 (2.79)
10	0.42 (1.38)	4.59	1.16 (3.81)
15	0.47 (1.54)	4.96	1.39 (4.56)
20	0.56 (1.84)	5.37	1.67 (5.48)
25	0.63 (2.07)	5.69	1.82 (5.97)
30	0.69 (2.26)	6.45	2.02 (6.63)
35	0.75 (2.46)	6.45	2.02 (6.63)
40	0.80 (2.62)	6.92	1.82 (5.97)
45	0.84 (2.76)	7.25	1.71 (5.61)
50	0.88 (2.89)	6.79	1.90 (6.23)
100	1.14 (3.74)	7.55	1.86 (6.10)

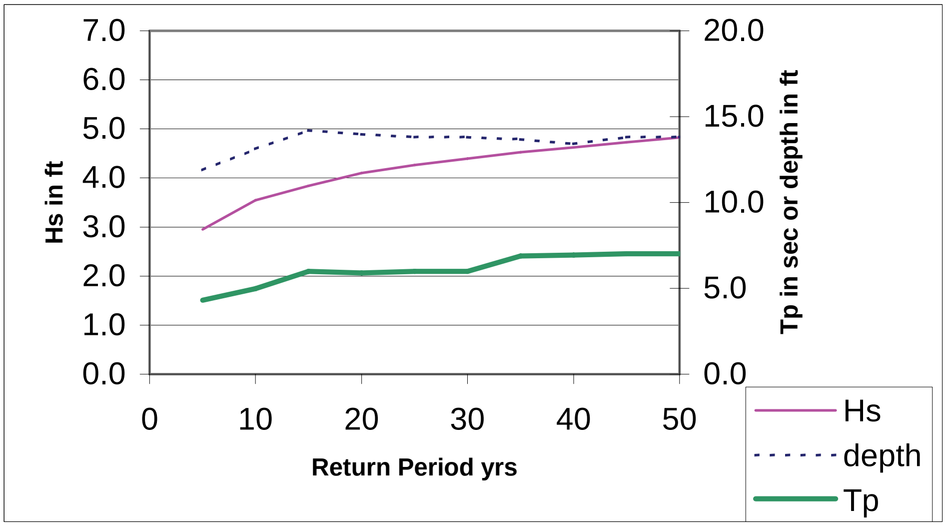

Figure B1. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 33 wave data

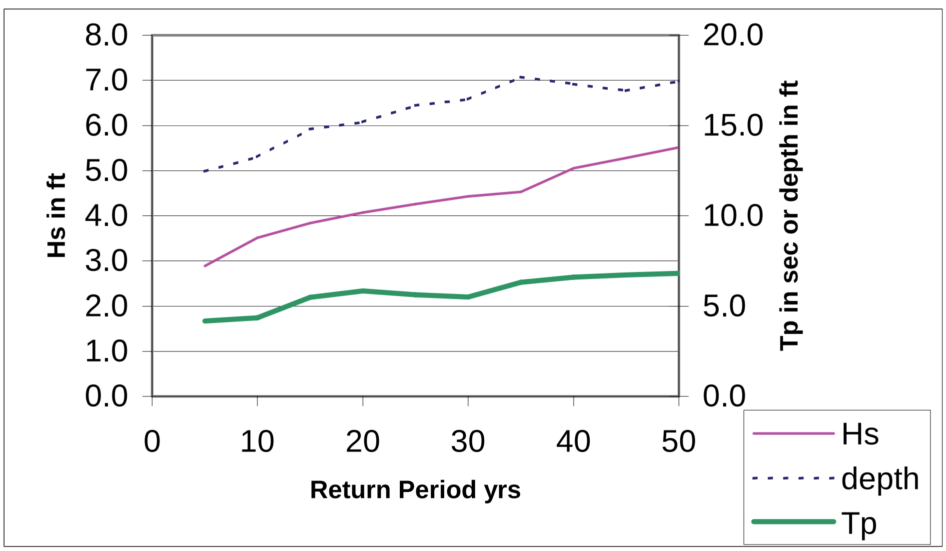

Figure B2. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 34 wave data

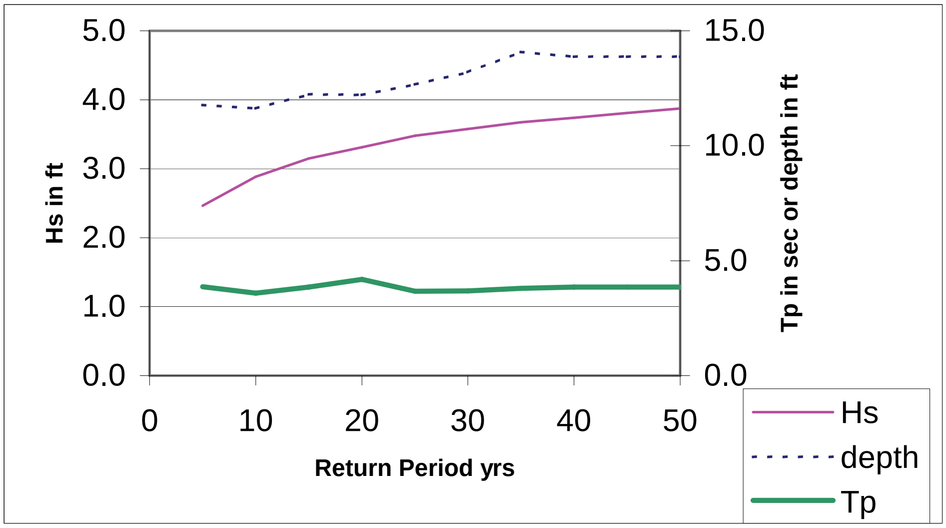

Figure B3. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 35 wave data

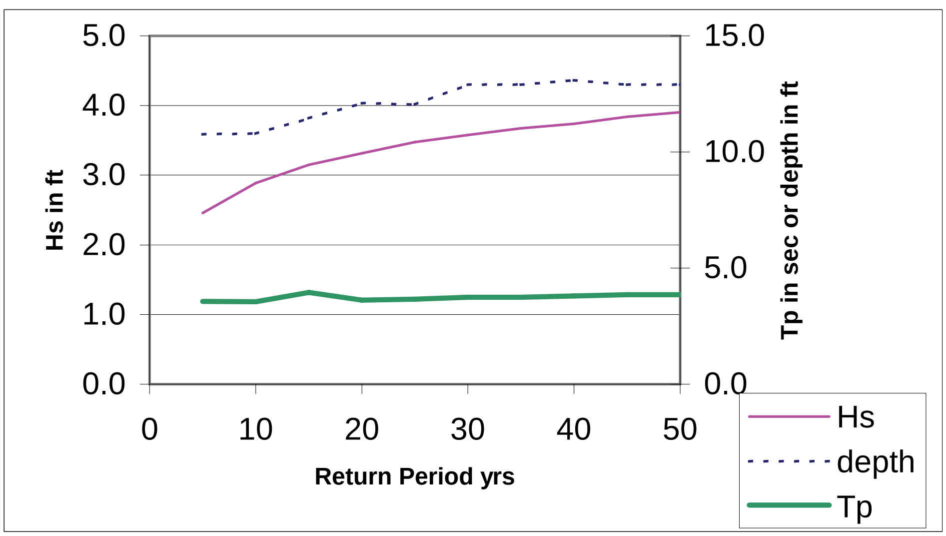

Figure B4. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 36 wave data

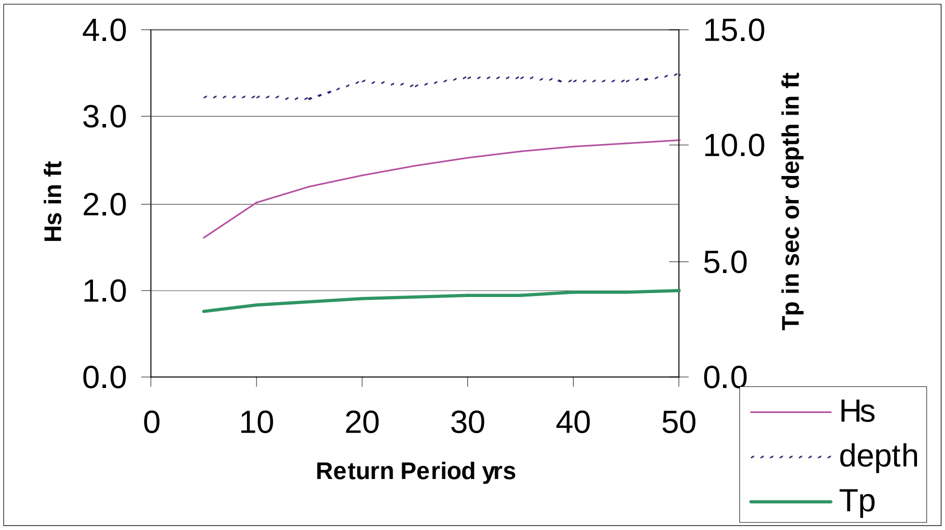

Figure B5. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 37 wave data

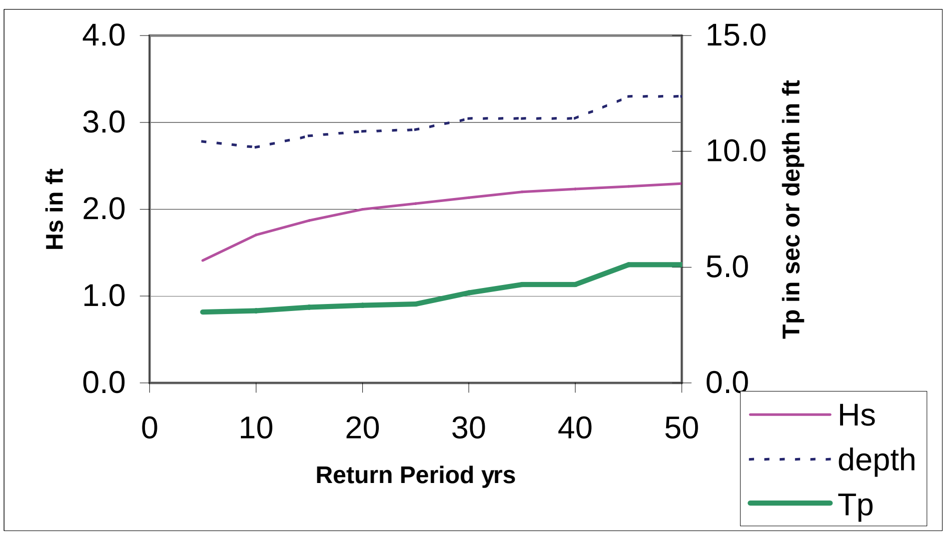

Figure B6. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 38 wave data

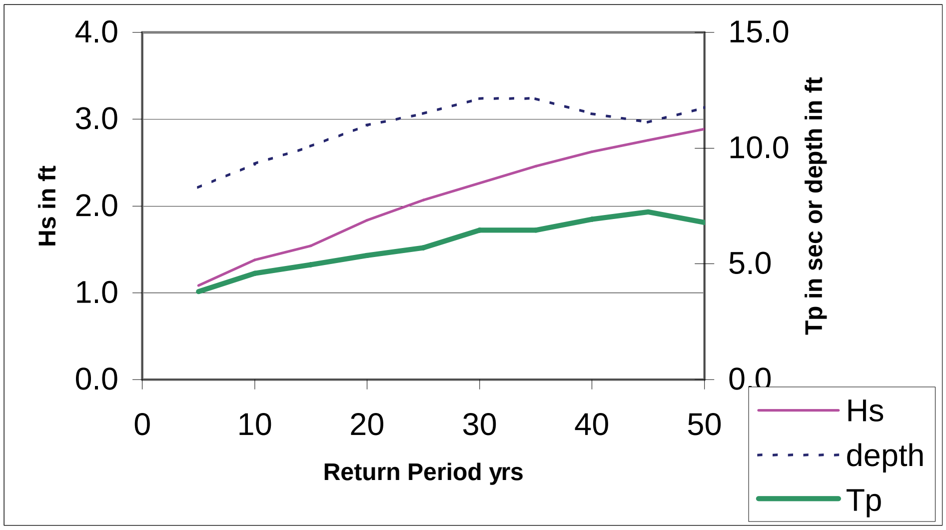

Figure B7. Significant wave height, peak wave period, and total depth as function of return period from extremal wave height analysis of sta 39 wave data

Table B24 **Extreme Water Levels for Historical Northeasters from Poplar Island Water Level Analysis Station 1 (Figure 19)**	
Return Period, years	**Water Level Relative to msl, ft**
5	2.21
10	2.6
25	3.1
50	3.46
100	3.83

Table B25 **Extreme Water Levels for Historical Hurricanes from Poplar Island Water Level Analysis Station 1 (Figure 19)**	
Return Period, years	**Water Level Relative to msl, ft**
5	2.02
10	3.06
25	4.38
50	5.35
100	6.32

Appendix C
Armor Weight as Function of Return Period for Poplar Island

Figures C1-C8 show the stable main armor weight as a function of return period for each design analysis station of the northern expansion of Poplar Island computed using Equation 23 (Hudson), Equations 24-26 (van der Meer), and Equations 30-34 (Melby and Hughes) and the extremal waves from Appendix B. In general, the results for a seaward structure slope of cot α = 3.0 are shown. The results for cot α = 2.5 for sta 33 are shown for comparison.

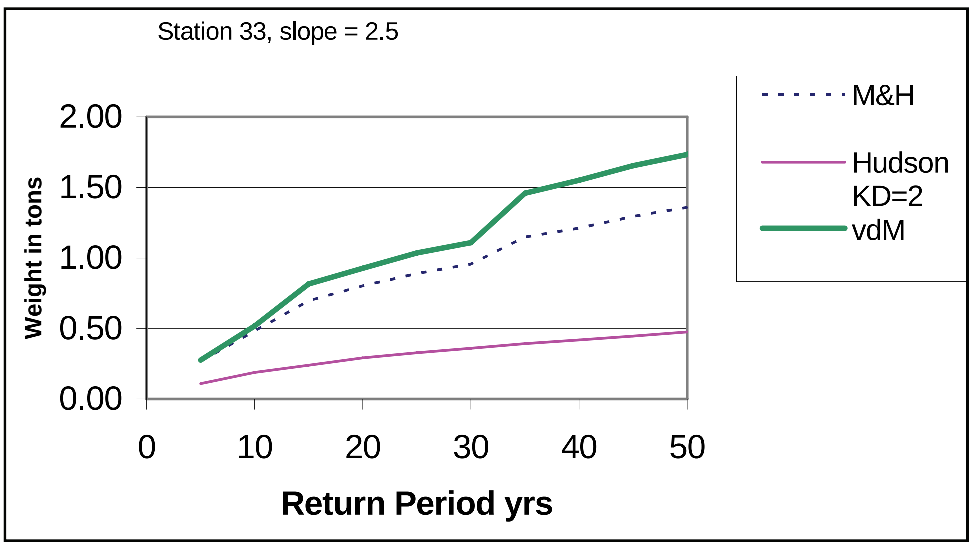

Figure C1. Armor weight as function of return period for Poplar Island sta 33 with structure slope of 1V:2.5H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

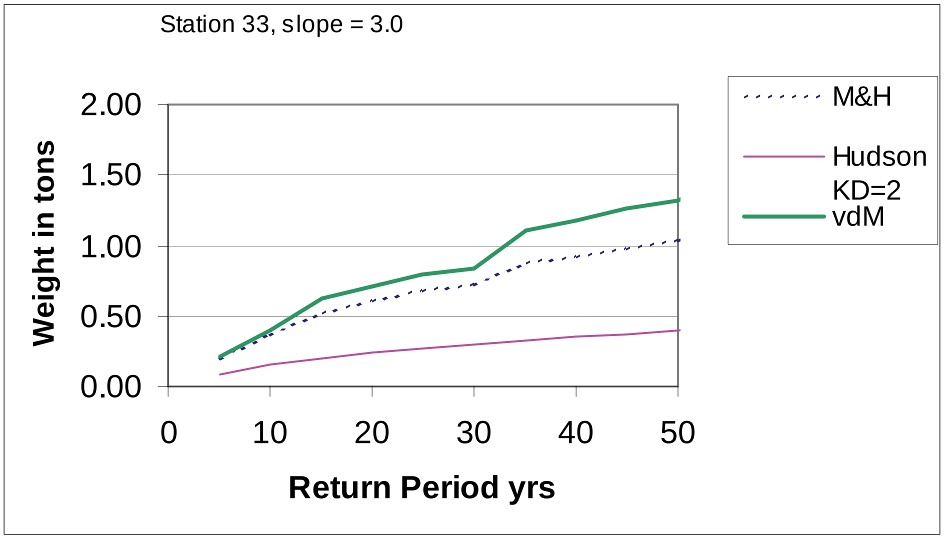

Figure C2. Armor weight as function of return period for Poplar Island sta 33 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

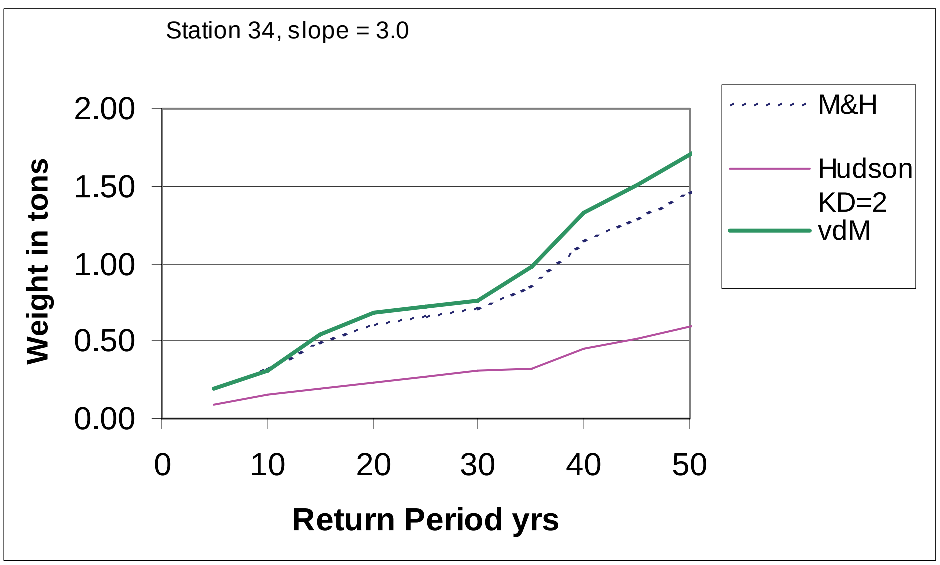

Figure C3. Armor weight as function of return period for Poplar Island sta 34 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

Figure C4. Armor weight as function of return period for Poplar Island sta 35 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

Figure C5. Armor weight as function of return period for Poplar Island sta 36 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

Figure C6. Armor weight as function of return period for Poplar Island sta 37 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

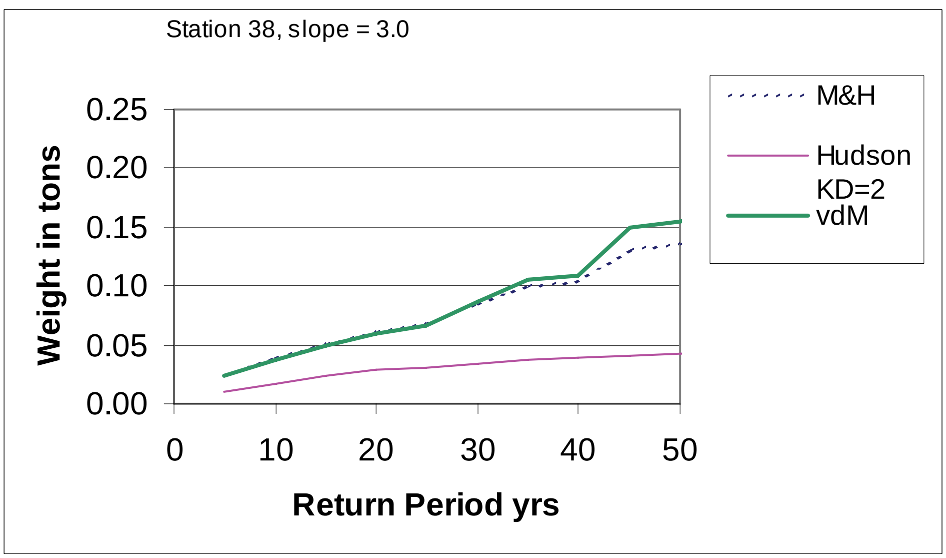

Figure C7. Armor weight as function of return period for Poplar Island sta 38 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

Figure C8. Armor weight as function of return period for Poplar Island sta 39 with structure slope of 1V:3.0H using stability equations from Melby and Hughes (M&H), Hudson, and van der Meer (vdM)

Appendix D
Maximum Significant Wave Height for Storm History for James Island

Maximum significant wave height by storm, needed to determine return-period wave height values for structure design, was extracted along with corresponding peak wave period, wave direction, and water level. Separate output files were created for tropical storms only, northeasters only, and all storms together. These values of maximum H_s for each storm as well as associated peak period, direction, and water level are tabulated for selected design analysis stations of James Island in this appendix. Tables D1-D7 summarize hurricanes and Tables D8-D14 summarize extratropical storms.

Table D1
Maximum H_S by Storm, James Island, Station 1, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/19	0.21 (0.69)	2.26	206.47	0.32 (1.05)	6
2	None	1861/09/28	0.57 (1.87)	4.70	209.72	0.97 (3.18)	30
3	None	1861/11/02	0.43 (1.41)	2.90	202.52	0.68 (2.23)	24
4	None	1863/09/19	0.60 (1.97)	3.40	211.21	0.70 (2.30)	21
5	None	1874/09/30	0.77 (2.53)	5.60	217.64	1.19 (3.90)	42
6	None	1876/09/18	0.88 (2.89)	6.39	217.31	1.34 (4.40)	54
7	None	1877/10/05	0.68 (2.23)	5.00	209.72	1.12 (3.67)	24
8	None	1878/10/23	1.10 (3.61)	5.50	213.49	1.54 (5.05)	39
9	None	1879/08/18	0.51 (1.67)	3.60	199.31	0.87 (2.85)	33
10	None	1880/09/09	0.41 (1.35)	3.10	207.73	0.46 (1.51)	21
11	None	1881/09/10	0.51 (1.67)	3.16	202.52	0.67 (2.20)	12
12	None	1888/10/12	0.51 (1.67)	3.20	205.00	0.56 (1.84)	33
13	None	1889/09/25	0.47 (1.54)	3.00	203.76	0.77 (2.53)	30
14	None	1893/06/17	0.52 (1.71)	3.19	202.52	0.68 (2.23)	21
15	None	1893/08/29	0.74 (2.43)	5.50	213.56	0.94 (3.08)	45
16	None	1893/10/14	0.76 (2.49)	5.00	213.04	1.16 (3.81)	39
17	None	1893/10/23	0.38 (1.25)	3.01	207.73	0.33 (1.08)	24
18	None	1894/09/28	0.39 (1.28)	2.75	201.27	0.66 (2.17)	36
19	None	1894/10/10	0.50 (1.64)	3.40	210.27	0.53 (1.74)	30
20	None	1897/10/24	0.12 (0.39)	1.72	206.47	0.27 (0.89)	0
21	None	1899/08/18	0.17 (0.56)	3.50	48.04	1.10 (3.61)	15
22	None	1899/11/01	0.96 (3.15)	4.50	213.27	1.38 (4.53)	33
23	None	1904/09/15	0.89 (2.92)	5.40	206.74	1.35 (4.43)	27
24	None	1908/08/01	0.07 (0.23)	2.96	46.90	0.57 (1.87)	0
25	None	1923/10/24	0.31 (1.02)	2.70	217.87	0.42 (1.38)	9
26	None	1933/08/24	0.87 (2.85)	6.09	219.93	1.36 (4.46)	18
27	None	1933/09/17	0.16 (0.52)	3.25	45.96	0.93 (3.05)	3
28	None	1935/09/06	0.51 (1.67)	3.20	205.00	0.57 (1.87)	18
29	None	1936/09/18	0.09 (0.30)	3.80	49.00	0.62 (2.03)	0
30	None	1944/08/03	0.70 (2.30)	4.60	211.27	0.87 (2.85)	42
31	None	1944/09/15	0.30 (0.98)	2.65	216.60	0.48 (1.57)	3
32	None	1946/07/07	0.28 (0.92)	2.56	207.73	0.49 (1.61)	12
33	Barbara	1953/08/15	0.20 (0.66)	2.19	216.60	0.45 (1.48)	3
34	Hazel	1954/10/16	0.98 (3.22)	8.29	222.98	1.33 (4.36)	21
35	Connie	1955/08/13	0.65 (2.13)	4.10	216.36	1.12 (3.67)	15
36	Diane	1955/08/19	0.56 (1.84)	4.30	219.45	0.64 (2.10)	33
37	Ione	1955/09/20	0.15 (0.49)	3.10	47.00	0.97 (3.18)	0
38	Brenda	1960/07/30	0.56 (1.84)	4.30	213.00	0.84 (2.76)	30
39	Donna	1960/09/12	0.49 (1.61)	3.10	202.52	0.71 (2.33)	39
40	Doria	1967/09/11	0.18 (0.59)	1.94	201.27	0.60 (1.97)	3
41	Doria	1971/08/28	0.46 (1.51)	3.00	211.21	0.67 (2.20)	27
42	Bret	1981/07/01	0.12 (0.39)	1.75	209.00	0.47 (1.54)	0
43	Dean	1983/09/30	0.03 (0.10)	2.26	46.00	0.39 (1.28)	0
44	Gloria	1985/09/27	0.25 (0.82)	3.10	237.00	0.85 (2.79)	3
45	Charley	1986/08/18	0.08 (0.26)	3.50	51.10	0.74 (2.43)	0
46	Danielle	1992/09/26	0.15 (0.49)	1.95	212.80	0.29 (0.95)	0
47	Bertha	1996/07/13	0.58 (1.90)	3.90	200.54	0.91 (2.99)	36
48	Fran	1996/09/07	0.88 (2.89)	4.30	208.25	1.20 (3.94)	45
49	Bonnie	1998/08/29	0.18 (0.59)	2.11	215.33	0.32 (1.05)	3
50	Earl	1998/09/04	0.53 (1.74)	3.25	206.24	0.58 (1.90)	33
51	Floyd	1999/09/16	0.60 (1.97)	3.40	202.52	0.71 (2.33)	24
52	Isabel	2003/09/19	1.04 (3.41)	5.22	209.75	1.83 (6.00)	18

[1]Storm duration is the time during a storm when $H_S > 0.15$ m.

Table D2
Maximum H_s by Storm, James Island, Station 3, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.32 (1.05)	3.40	294.00	0.34 (1.12)	3
2	None	1861/09/28	0.85 (2.79)	4.70	210.72	0.97 (3.18)	21
3	None	1861/11/02	0.38 (1.25)	2.90	195.60	0.68 (2.23)	9
4	None	1863/09/19	0.52 (1.71)	3.40	204.00	0.70 (2.30)	9
5	None	1874/09/29	1.32 (4.33)	5.95	215.30	1.15 (3.77)	27
6	None	1876/09/18	1.48 (4.86)	6.39	220.33	1.34 (4.40)	33
7	None	1877/10/05	1.02 (3.35)	5.00	210.72	1.12 (3.67)	18
8	None	1878/10/23	2.00 (6.56)	7.81	219.93	1.47 (4.82)	36
9	None	1879/08/18	0.60 (1.97)	3.60	191.45	0.87 (2.85)	30
10	None	1880/09/09	0.41 (1.35)	3.10	200.78	0.46 (1.51)	9
11	None	1881/09/10	0.47 (1.54)	3.34	199.55	0.52 (1.71)	12
12	None	1888/10/12	0.45 (1.48)	3.20	198.00	0.56 (1.84)	18
13	None	1889/09/25	0.48 (1.57)	3.30	203.22	0.37 (1.21)	21
14	None	1893/06/17	0.45 (1.48)	3.19	195.60	0.68 (2.23)	6
15	None	1893/08/29	1.57 (5.15)	7.00	228.41	0.76 (2.49)	39
16	None	1893/10/14	1.43 (4.69)	6.39	225.02	0.94 (3.08)	27
17	None	1893/10/23	0.59 (1.94)	3.80	207.71	0.35 (1.15)	9
18	None	1894/09/30	0.45 (1.48)	3.90	292.00	0.05 (0.16)	24
19	None	1894/10/10	0.59 (1.94)	3.60	197.79	0.61 (2.00)	21
20	None	1897/10/24	0.12 (0.39)	1.72	199.55	0.27 (0.89)	0
21	None	1899/08/19	0.40 (1.31)	3.70	294.00	0.38 (1.25)	9
22	None	1899/11/01	1.82 (5.97)	7.00	224.31	1.17 (3.84)	33
23	None	1904/09/15	1.15 (3.77)	5.40	206.74	1.35 (4.43)	27
24	None	1908/08/01	0.28 (0.92)	3.20	294.00	0.38 (1.25)	0
25	None	1923/10/24	0.48 (1.57)	2.70	258.10	0.35 (1.15)	6
26	None	1933/08/24	1.44 (4.72)	6.09	222.98	1.36 (4.46)	15
27	None	1933/09/17	0.40 (1.31)	3.70	296.00	0.65 (2.13)	6
28	None	1935/09/06	0.45 (1.48)	3.20	198.00	0.57 (1.87)	9
29	None	1936/09/19	0.62 (2.03)	4.60	286.00	0.43 (1.41)	6
30	None	1944/08/03	0.90 (2.95)	4.80	213.28	1.11 (3.64)	33
31	None	1944/09/15	0.35 (1.15)	2.34	262.00	0.81 (2.66)	6
32	None	1946/07/07	0.28 (0.92)	2.56	200.78	0.49 (1.61)	0
33	Barbara	1953/08/15	0.41 (1.35)	3.80	294.00	0.52 (1.71)	3
34	Hazel	1954/10/16	1.91 (6.27)	8.29	226.04	1.33 (4.36)	21
35	Connie	1955/08/13	0.73 (2.39)	4.10	216.36	1.12 (3.67)	12
36	Diane	1955/08/18	0.99 (3.25)	5.00	221.24	0.78 (2.56)	30
37	Ione	1955/09/20	0.46 (1.51)	3.95	294.00	0.39 (1.28)	3
38	Brenda	1960/07/30	0.76 (2.49)	4.30	213.00	0.84 (2.76)	18
39	Donna	1960/09/13	0.46 (1.51)	2.70	266.85	0.78 (2.56)	18
40	Doria	1967/09/11	0.16 (0.52)	1.94	194.40	0.60 (1.97)	0
41	Doria	1971/08/28	0.41 (1.35)	3.00	204.00	0.67 (2.20)	18
42	Bret	1981/07/01	0.12 (0.39)	1.75	202.00	0.47 (1.54)	0
43	Dean	1983/09/30	0.02 (0.07)	2.26	320.00	0.39 (1.28)	0
44	Gloria	1985/09/27	0.83 (2.72)	5.20	287.00	0.81 (2.66)	6
45	Charley	1986/08/18	0.35 (1.15)	3.50	294.00	0.33 (1.08)	6
46	Danielle	1992/09/26	0.15 (0.49)	1.95	205.67	0.29 (0.95)	0
47	Bertha	1996/07/13	0.66 (2.17)	3.10	261.03	0.90 (2.95)	27
48	Fran	1996/09/07	0.85 (2.79)	4.30	209.26	1.20 (3.94)	27
49	Bonnie	1998/08/29	0.18 (0.59)	2.61	296.00	0.62 (2.03)	0
50	Earl	1998/09/04	0.48 (1.57)	3.25	199.20	0.58 (1.90)	12
51	Floyd	1999/09/17	0.71 (2.33)	3.23	279.47	1.16 (3.81)	24
52	Isabel	2003/09/19	1.50 (4.92)	6.20	218.62	1.85 (6.07)	18

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table D3
Maximum H_s by Storm, James Island, Station 5, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.56 (1.84)	3.40	311.00	0.34 (1.12)	15
2	None	1861/09/28	0.76 (2.49)	4.70	212.70	0.96 (3.15)	18
3	None	1861/11/03	0.41 (1.35)	2.90	313.00	0.67 (2.20)	12
4	None	1863/09/19	0.46 (1.51)	3.10	313.00	0.59 (1.94)	15
5	None	1874/09/29	1.19 (3.90)	5.95	215.30	1.14 (3.74)	24
6	None	1876/09/18	1.31 (4.30)	6.39	221.33	1.35 (4.43)	24
7	None	1877/10/05	1.09 (3.58)	3.80	263.02	1.02 (3.35)	15
8	None	1878/10/23	2.00 (6.56)	7.81	222.98	1.49 (4.89)	42
9	None	1879/08/19	0.58 (1.90)	3.45	313.00	0.86 (2.82)	27
10	None	1880/09/09	0.36 (1.18)	3.10	203.76	0.45 (1.48)	9
11	None	1881/09/11	0.48 (1.57)	3.20	313.00	0.60 (1.97)	18
12	None	1888/10/12	0.39 (1.28)	3.20	202.00	0.57 (1.87)	12
13	None	1889/09/25	0.42 (1.38)	3.30	206.24	0.37 (1.21)	15
14	None	1893/06/17	0.46 (1.51)	3.15	311.00	0.40 (1.31)	15
15	None	1893/08/29	1.49 (4.89)	7.00	229.43	0.77 (2.53)	30
16	None	1893/10/14	1.27 (4.17)	6.39	226.04	0.96 (3.15)	21
17	None	1893/10/23	0.53 (1.74)	3.80	208.73	0.35 (1.15)	15
18	None	1894/09/30	0.76 (2.49)	3.90	309.00	0.06 (0.20)	27
19	None	1894/10/10	0.59 (1.94)	3.50	313.00	0.77 (2.53)	21
20	None	1897/10/26	0.45 (1.48)	3.50	336.14	0.06 (0.20)	12
21	None	1899/08/19	0.68 (2.23)	3.70	311.00	0.38 (1.25)	63
22	None	1899/11/01	1.65 (5.41)	7.00	225.33	1.19 (3.90)	24
23	None	1904/09/15	1.20 (3.94)	3.90	267.01	1.23 (4.04)	24
24	None	1908/08/01	0.48 (1.57)	3.20	311.00	0.39 (1.28)	9
25	None	1923/10/24	0.53 (1.74)	2.70	266.01	0.34 (1.12)	12
26	None	1933/08/24	1.29 (4.23)	6.09	224.00	1.38 (4.53)	30
27	None	1933/09/17	0.69 (2.26)	3.70	313.00	0.66 (2.17)	24
28	None	1935/09/06	0.39 (1.28)	3.20	202.00	0.57 (1.87)	9
29	None	1936/09/19	1.00 (3.28)	4.60	299.00	0.43 (1.41)	33
30	None	1944/08/03	0.80 (2.62)	4.80	215.30	1.11 (3.64)	33
31	None	1944/09/15	0.49 (1.61)	3.80	338.87	0.57 (1.87)	18
32	None	1946/07/07	0.25 (0.82)	2.56	203.76	0.49 (1.61)	0
33	Barbara	1953/08/15	0.70 (2.30)	3.80	311.00	0.52 (1.71)	21
34	Hazel	1954/10/16	2.11 (6.92)	8.29	228.07	1.36 (4.46)	21
35	Connie	1955/08/13	0.82 (2.69)	5.00	336.00	1.16 (3.81)	33
36	Diane	1955/08/18	0.90 (2.95)	5.00	221.24	0.80 (2.62)	30
37	Ione	1955/09/20	0.78 (2.56)	3.95	311.00	0.39 (1.28)	21
38	Brenda	1960/07/30	0.67 (2.20)	4.30	216.00	0.84 (2.76)	12
39	Donna	1960/09/12	0.67 (2.20)	3.70	313.00	0.86 (2.82)	21
40	Doria	1967/09/11	0.14 (0.46)	1.94	198.33	0.61 (2.00)	0
41	Doria	1971/08/28	0.58 (1.90)	3.45	315.00	0.98 (3.22)	18
42	Bret	1981/07/01	0.46 (1.51)	3.40	24.53	0.46 (1.51)	9
43	Dean	1983/09/30	0.20 (0.66)	2.26	337.00	0.40 (1.31)	0
44	Gloria	1985/09/27	1.14 (3.74)	5.20	298.00	0.79 (2.59)	15
45	Charley	1986/08/18	0.60 (1.97)	3.50	311.00	0.33 (1.08)	15
46	Danielle	1992/09/26	0.25 (0.82)	2.58	338.73	0.45 (1.48)	0
47	Bertha	1996/07/13	0.71 (2.33)	3.10	268.00	0.91 (2.99)	15
48	Fran	1996/09/07	0.73 (2.39)	4.30	213.28	1.21 (3.97)	27
49	Bonnie	1998/08/29	0.32 (1.05)	2.61	313.00	0.62 (2.03)	3
50	Earl	1998/09/04	0.41 (1.35)	3.25	203.22	0.58 (1.90)	9
51	Floyd	1999/09/17	0.77 (2.53)	3.23	286.93	1.15 (3.77)	30
52	Isabel	2003/09/19	1.32 (4.33)	6.20	221.65	1.87 (6.14)	30

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table D4
Maximum H_s by Storm, James Island, Station 7, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.61 (2.00)	3.40	319.00	0.34 (1.12)	21
2	None	1861/09/28	0.58 (1.90)	4.70	229.60	0.96 (3.15)	6
3	None	1861/11/03	0.43 (1.41)	2.90	319.00	0.67 (2.20)	15
4	None	1863/09/19	0.49 (1.61)	3.10	319.00	0.59 (1.94)	6
5	None	1874/09/29	1.00 (3.28)	5.95	232.40	1.14 (3.74)	12
6	None	1876/09/18	1.09 (3.58)	6.39	234.41	1.35 (4.43)	15
7	None	1877/10/05	1.07 (3.51)	3.80	267.91	1.02 (3.35)	15
8	None	1878/10/23	1.70 (5.58)	7.81	236.22	1.49 (4.89)	24
9	None	1879/08/19	0.62 (2.03)	3.45	319.00	0.86 (2.82)	12
10	None	1880/09/10	0.30 (0.98)	2.80	350.00	0.73 (2.39)	0
11	None	1881/09/11	0.51 (1.67)	3.20	319.00	0.60 (1.97)	12
12	None	1888/10/12	0.22 (0.72)	3.20	221.00	0.57 (1.87)	0
13	None	1889/09/25	0.29 (0.95)	2.80	350.00	0.57 (1.87)	0
14	None	1893/06/17	0.50 (1.64)	3.15	319.00	0.40 (1.31)	15
15	None	1893/08/29	1.37 (4.49)	7.00	241.72	0.77 (2.53)	21
16	None	1893/10/14	1.09 (3.58)	6.39	239.27	0.96 (3.15)	12
17	None	1893/10/23	0.44 (1.44)	3.50	349.10	0.59 (1.94)	9
18	None	1894/09/30	0.84 (2.76)	3.90	319.00	0.06 (0.20)	30
19	None	1894/10/10	0.62 (2.03)	3.50	319.00	0.77 (2.53)	12
20	None	1897/10/26	0.51 (1.67)	3.50	346.11	0.06 (0.20)	12
21	None	1899/08/19	0.74 (2.43)	3.70	319.00	0.38 (1.25)	69
22	None	1899/11/01	1.43 (4.69)	7.00	238.65	1.19 (3.90)	12
23	None	1904/09/15	1.18 (3.87)	3.90	271.97	1.23 (4.04)	12
24	None	1908/08/01	0.52 (1.71)	3.20	319.00	0.39 (1.28)	15
25	None	1923/10/24	0.56 (1.84)	3.80	347.11	0.51 (1.67)	12
26	None	1933/08/24	1.05 (3.44)	6.09	237.24	1.38 (4.53)	27
27	None	1933/09/17	0.73 (2.39)	3.70	319.00	0.66 (2.17)	27
28	None	1935/09/06	0.35 (1.15)	2.80	40.26	0.64 (2.10)	6
29	None	1936/09/19	1.10 (3.61)	4.60	314.00	0.43 (1.41)	36
30	None	1944/08/03	0.67 (2.20)	3.10	293.28	0.97 (3.18)	21
31	None	1944/09/15	0.53 (1.74)	3.80	350.90	0.57 (1.87)	15
32	None	1946/07/07	0.25 (0.82)	2.24	319.00	0.47 (1.54)	0
33	Barbara	1953/08/15	0.77 (2.53)	3.80	319.00	0.52 (1.71)	21
34	Hazel	1954/10/16	1.93 (6.33)	8.29	240.29	1.36 (4.46)	21
35	Connie	1955/08/13	0.91 (2.99)	5.00	345.00	1.16 (3.81)	30
36	Diane	1955/08/18	0.77 (2.53)	5.00	238.65	0.80 (2.62)	21
37	Ione	1955/09/20	0.85 (2.79)	3.95	319.00	0.39 (1.28)	24
38	Brenda	1960/07/30	0.49 (1.61)	4.30	233.00	0.84 (2.76)	6
39	Donna	1960/09/12	0.71 (2.33)	3.70	319.00	0.86 (2.82)	12
40	Doria	1967/09/11	0.08 (0.26)	1.39	350.68	0.28 (0.92)	0
41	Doria	1971/08/28	0.61 (2.00)	3.45	320.00	0.98 (3.22)	9
42	Bret	1981/07/01	0.50 (1.64)	3.40	37.08	0.46 (1.51)	12
43	Dean	1983/09/30	0.21 (0.69)	2.26	348.00	0.40 (1.31)	0
44	Gloria	1985/09/27	1.26 (4.13)	5.20	312.00	0.79 (2.59)	18
45	Charley	1986/08/18	0.66 (2.17)	3.50	319.00	0.33 (1.08)	15
46	Danielle	1992/09/26	0.27 (0.89)	2.58	349.78	0.45 (1.48)	0
47	Bertha	1996/07/13	0.70 (2.30)	3.10	272.99	0.91 (2.99)	9
48	Fran	1996/09/06	0.50 (1.64)	4.35	232.40	1.13 (3.71)	21
49	Bonnie	1998/08/29	0.34 (1.12)	2.61	319.00	0.62 (2.03)	9
50	Earl	1998/09/04	0.24 (0.79)	3.25	222.34	0.58 (1.90)	0
51	Floyd	1999/09/17	0.76 (2.49)	3.23	292.27	1.15 (3.77)	15
52	Isabel	2003/09/19	1.05 (3.44)	6.20	234.81	1.87 (6.14)	27

[1]Storm duration is the time during a storm when H_S > 0.3 m.

Table D5
Maximum H_s by Storm, James Island, Station 9, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.54 (1.77)	3.25	20.97	0.69 (2.26)	18
2	None	1861/09/26	0.01 (0.03)	0.74	276.67	0.31 (1.02)	0
3	None	1861/11/03	0.50 (1.64)	3.10	21.95	0.58 (1.90)	12
4	None	1863/09/19	0.59 (1.94)	3.25	73.59	0.65 (2.13)	6
5	None	1874/09/28	0.01 (0.03)	1.47	278.33	0.31 (1.02)	0
6	None	1876/09/19	0.11 (0.36)	1.64	299.50	0.74 (2.43)	0
7	None	1877/10/05	0.71 (2.33)	3.80	296.27	1.10 (3.61)	6
8	None	1878/10/24	0.23 (0.75)	2.30	286.17	1.51 (4.95)	0
9	None	1879/08/19	0.66 (2.17)	3.80	21.95	0.84 (2.76)	6
10	None	1880/09/10	0.43 (1.41)	2.80	20.00	0.74 (2.43)	6
11	None	1881/09/10	0.46 (1.51)	2.90	20.00	0.87 (2.85)	15
12	None	1888/10/12	0.28 (0.92)	2.30	17.93	0.54 (1.77)	0
13	None	1889/09/25	0.42 (1.38)	2.80	20.00	0.59 (1.94)	6
14	None	1893/06/17	0.43 (1.41)	2.90	14.07	0.47 (1.54)	15
15	None	1893/08/27	0.18 (0.59)	1.80	69.03	0.32 (1.05)	0
16	None	1893/10/12	0.16 (0.52)	1.72	17.93	0.31 (1.02)	0
17	None	1893/10/23	0.58 (1.90)	3.45	21.95	0.59 (1.94)	6
18	None	1894/09/30	0.69 (2.26)	3.90	326.00	0.07 (0.23)	36
19	None	1894/10/10	0.54 (1.77)	3.30	22.92	0.71 (2.33)	6
20	None	1897/10/25	0.55 (1.80)	3.50	15.04	0.33 (1.08)	18
21	None	1899/08/18	0.65 (2.13)	3.50	24.98	1.12 (3.67)	78
22	None	1899/10/31	0.24 (0.79)	2.03	69.99	0.28 (0.92)	0
23	None	1904/09/15	0.79 (2.59)	3.90	300.76	1.31 (4.30)	3
24	None	1908/08/01	0.46 (1.51)	2.96	18.05	0.61 (2.00)	24
25	None	1923/10/24	0.63 (2.07)	3.80	15.04	0.52 (1.71)	15
26	None	1933/08/24	0.70 (2.30)	3.70	26.95	0.88 (2.89)	15
27	None	1933/09/17	0.61 (2.00)	3.70	323.00	0.69 (2.26)	45
28	None	1935/09/06	0.50 (1.64)	2.80	74.56	0.63 (2.07)	6
29	None	1936/09/19	0.94 (3.08)	4.60	330.00	0.44 (1.44)	39
30	None	1944/08/03	0.43 (1.41)	3.10	324.32	1.03 (3.38)	6
31	None	1944/09/15	0.66 (2.17)	3.80	20.97	0.56 (1.84)	18
32	None	1946/07/07	0.32 (1.05)	2.44	17.93	0.42 (1.38)	3
33	Barbara	1953/08/15	0.64 (2.10)	3.80	325.00	0.55 (1.80)	27
34	Hazel	1954/10/15	0.53 (1.74)	3.15	69.99	0.47 (1.54)	6
35	Connie	1955/08/13	0.96 (3.15)	5.00	15.00	1.11 (3.64)	33
36	Diane	1955/08/17	0.34 (1.12)	2.38	74.56	0.58 (1.90)	12
37	Ione	1955/09/20	0.71 (2.33)	3.95	325.00	0.39 (1.28)	45
38	Brenda	1960/07/30	0.42 (1.38)	2.70	24.00	0.89 (2.92)	3
39	Donna	1960/09/12	0.68 (2.23)	3.35	78.15	0.83 (2.72)	12
40	Doria	1967/09/11	0.10 (0.33)	1.39	18.89	0.28 (0.92)	0
41	Doria	1971/08/28	0.70 (2.30)	3.40	78.15	0.89 (2.92)	12
42	Bret	1981/07/01	0.59 (1.94)	3.40	69.03	0.40 (1.31)	24
43	Dean	1983/09/30	0.27 (0.89)	2.26	16.00	0.42 (1.38)	0
44	Gloria	1985/09/27	1.22 (4.00)	5.20	328.00	0.78 (2.56)	30
45	Charley	1986/08/18	0.61 (2.00)	3.50	21.95	0.73 (2.39)	18
46	Danielle	1992/09/26	0.35 (1.15)	2.58	17.93	0.42 (1.38)	6
47	Bertha	1996/07/13	0.63 (2.07)	3.40	26.95	1.15 (3.77)	6
48	Fran	1996/09/06	0.53 (1.74)	3.00	73.59	0.59 (1.94)	12
49	Bonnie	1998/08/28	0.52 (1.71)	3.00	24.00	0.94 (3.08)	60
50	Earl	1998/09/02	0.01 (0.03)	0.92	280.00	0.31 (1.02)	0
51	Floyd	1999/09/17	0.76 (2.49)	4.25	14.04	1.15 (3.77)	9
52	Isabel	2003/09/19	0.82 (2.69)	4.06	67.88	0.92 (3.02)	18

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table D6
Maximum H_s by Storm, James Island, Station 11, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/20	0.40 (1.31)	3.25	25.99	0.69 (2.26)	27
2	None	1861/09/28	0.16 (0.52)	3.10	155.45	1.10 (3.61)	3
3	None	1861/11/03	0.37 (1.21)	3.10	26.97	0.58 (1.90)	9
4	None	1863/09/19	0.43 (1.41)	3.25	79.29	0.65 (2.13)	6
5	None	1874/09/30	0.28 (0.92)	3.10	160.15	1.29 (4.23)	3
6	None	1876/09/18	0.16 (0.52)	3.10	153.64	1.08 (3.54)	3
7	None	1877/10/05	0.20 (0.66)	3.80	314.84	1.10 (3.61)	3
8	None	1878/10/24	0.34 (1.12)	3.30	158.45	1.83 (6.00)	12
9	None	1879/08/19	0.50 (1.64)	3.80	26.97	0.84 (2.76)	6
10	None	1880/09/10	0.32 (1.05)	2.80	25.00	0.74 (2.43)	6
11	None	1881/09/10	0.33 (1.08)	2.90	25.00	0.87 (2.85)	33
12	None	1888/10/12	0.20 (0.66)	2.30	24.96	0.54 (1.77)	3
13	None	1889/09/25	0.30 (0.98)	2.80	25.00	0.59 (1.94)	6
14	None	1893/06/17	0.30 (0.98)	2.90	21.04	0.47 (1.54)	15
15	None	1893/08/29	0.16 (0.52)	3.70	159.49	1.39 (4.56)	3
16	None	1893/10/14	0.14 (0.46)	2.80	152.73	1.03 (3.38)	0
17	None	1893/10/23	0.43 (1.41)	3.45	26.97	0.59 (1.94)	9
18	None	1894/09/29	0.45 (1.48)	3.50	24.01	0.61 (2.00)	39
19	None	1894/10/10	0.40 (1.31)	3.30	27.96	0.71 (2.33)	6
20	None	1897/10/25	0.40 (1.31)	3.50	22.02	0.33 (1.08)	21
21	None	1899/08/18	0.67 (2.20)	3.50	37.02	1.12 (3.67)	96
22	None	1899/11/01	0.28 (0.92)	3.23	156.49	1.33 (4.36)	12
23	None	1904/09/15	0.29 (0.95)	3.30	157.41	1.21 (3.97)	18
24	None	1908/08/01	0.34 (1.12)	2.96	23.03	0.61 (2.00)	45
25	None	1923/10/24	0.46 (1.51)	3.80	22.02	0.52 (1.71)	18
26	None	1933/08/24	0.71 (2.33)	3.70	39.05	0.88 (2.89)	18
27	None	1933/09/17	0.60 (1.97)	3.25	34.98	0.92 (3.02)	66
28	None	1935/09/06	0.37 (1.21)	2.80	80.28	0.63 (2.07)	6
29	None	1936/09/18	0.50 (1.64)	3.80	25.00	0.59 (1.94)	57
30	None	1944/08/03	0.15 (0.49)	3.50	155.78	1.26 (4.13)	0
31	None	1944/09/15	0.50 (1.64)	3.80	25.99	0.56 (1.84)	36
32	None	1946/07/07	0.23 (0.75)	2.44	24.96	0.42 (1.38)	12
33	Barbara	1953/08/14	0.43 (1.41)	3.40	25.00	0.56 (1.84)	39
34	Hazel	1954/10/15	0.38 (1.25)	3.15	77.99	0.47 (1.54)	30
35	Connie	1955/08/13	0.98 (3.22)	5.00	38.00	1.11 (3.64)	78
36	Diane	1955/08/17	0.25 (0.82)	2.38	80.28	0.58 (1.90)	39
37	Ione	1955/09/20	0.55 (1.80)	3.10	36.00	0.98 (3.22)	54
38	Brenda	1960/07/30	0.43 (1.41)	2.70	36.00	0.89 (2.92)	3
39	Donna	1960/09/12	0.70 (2.30)	3.35	91.85	0.83 (2.72)	9
40	Doria	1967/09/11	0.07 (0.23)	1.39	25.95	0.28 (0.92)	0
41	Doria	1971/08/28	0.72 (2.36)	3.40	91.85	0.89 (2.92)	12
42	Bret	1981/07/01	0.43 (1.41)	3.40	77.01	0.40 (1.31)	33
43	Dean	1983/09/30	0.19 (0.62)	2.26	23.00	0.42 (1.38)	6
44	Gloria	1985/09/27	0.57 (1.87)	4.60	26.99	0.73 (2.39)	57
45	Charley	1986/08/18	0.45 (1.48)	3.50	26.97	0.73 (2.39)	24
46	Danielle	1992/09/26	0.25 (0.82)	2.58	24.96	0.42 (1.38)	15
47	Bertha	1996/07/13	0.65 (2.13)	3.40	39.05	1.15 (3.77)	3
48	Fran	1996/09/06	0.38 (1.25)	3.00	79.29	0.59 (1.94)	42
49	Bonnie	1998/08/28	0.53 (1.74)	3.00	36.00	0.94 (3.08)	72
50	Earl	1998/09/04	0.06 (0.20)	3.25	151.92	0.59 (1.94)	0
51	Floyd	1999/09/17	0.76 (2.49)	4.25	36.98	1.15 (3.77)	33
52	Isabel	2003/09/19	0.82 (2.69)	4.06	94.13	0.92 (3.02)	42

[1]Storm duration is the time during a storm when $H_S > 0.15$ m.

Table D7
Maximum H_s by Storm, James Island, Station 13, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	None	1856/08/19	0.04 (0.13)	2.26	187.70	0.31 (1.02)	0
2	None	1861/09/28	0.20 (0.66)	3.10	175.15	1.09 (3.58)	12
3	None	1861/11/02	0.14 (0.46)	2.90	198.56	0.66 (2.17)	0
4	None	1863/09/19	0.18 (0.59)	3.40	207.09	0.68 (2.23)	3
5	None	1874/09/30	0.28 (0.92)	3.10	159.09	1.29 (4.23)	12
6	None	1876/09/18	0.26 (0.85)	5.30	196.51	1.34 (4.40)	15
7	None	1877/10/05	0.19 (0.62)	5.00	203.76	1.00 (3.28)	3
8	None	1878/10/23	0.39 (1.28)	7.81	189.38	1.19 (3.90)	24
9	None	1879/08/18	0.16 (0.52)	3.60	188.51	0.80 (2.62)	3
10	None	1880/09/09	0.07 (0.23)	3.10	188.85	0.40 (1.31)	0
11	None	1881/09/10	0.16 (0.52)	3.16	198.56	0.70 (2.30)	3
12	None	1888/10/12	0.16 (0.52)	3.20	201.00	0.58 (1.90)	3
13	None	1889/09/25	0.15 (0.49)	3.00	199.78	0.76 (2.49)	0
14	None	1893/06/17	0.16 (0.52)	3.19	198.56	0.67 (2.20)	3
15	None	1893/08/29	0.23 (0.75)	3.70	179.30	1.38 (4.53)	12
16	None	1893/10/14	0.23 (0.75)	5.00	197.68	1.18 (3.87)	18
17	None	1893/10/24	0.07 (0.23)	3.10	192.30	0.32 (1.05)	0
18	None	1894/09/28	0.12 (0.39)	2.75	197.35	0.66 (2.17)	0
19	None	1894/10/10	0.11 (0.36)	3.60	200.78	0.55 (1.80)	0
20	None	1897/10/24	0.02 (0.07)	1.72	187.70	0.29 (0.95)	0
21	None	1899/08/18	0.10 (0.33)	3.50	17.97	1.13 (3.71)	0
22	None	1899/11/01	0.31 (1.02)	4.50	195.76	1.43 (4.69)	15
23	None	1904/09/15	0.30 (0.98)	3.30	156.36	1.21 (3.97)	12
24	None	1908/08/01	0.01 (0.03)	2.96	86.70	0.62 (2.03)	0
25	None	1923/10/24	0.05 (0.16)	2.70	198.06	0.40 (1.31)	0
26	None	1933/08/24	0.25 (0.82)	5.00	197.68	1.31 (4.30)	15
27	None	1933/09/17	0.09 (0.30)	3.25	16.03	0.94 (3.08)	0
28	None	1935/09/06	0.16 (0.52)	3.20	201.00	0.55 (1.80)	3
29	None	1936/09/18	0.01 (0.03)	3.80	89.00	0.61 (2.00)	0
30	None	1944/08/03	0.23 (0.75)	4.60	205.24	0.79 (2.59)	12
31	None	1944/09/15	0.05 (0.16)	2.65	196.91	0.47 (1.54)	0
32	None	1946/07/07	0.05 (0.16)	2.56	188.85	0.48 (1.57)	0
33	Barbara	1953/08/15	0.03 (0.10)	2.19	196.91	0.44 (1.44)	0
34	Hazel	1954/10/16	0.34 (1.12)	8.29	203.64	0.93 (3.05)	21
35	Connie	1955/08/14	0.21 (0.69)	3.20	173.10	1.00 (3.28)	9
36	Diane	1955/08/19	0.17 (0.56)	3.20	210.75	0.58 (1.90)	3
37	Ione	1955/09/20	0.08 (0.26)	3.10	17.00	0.99 (3.25)	0
38	Brenda	1960/07/30	0.13 (0.43)	4.30	205.00	0.79 (2.59)	0
39	Donna	1960/09/12	0.15 (0.49)	3.10	198.56	0.66 (2.17)	0
40	Doria	1967/09/11	0.06 (0.20)	1.94	197.35	0.62 (2.03)	0
41	Doria	1971/08/28	0.15 (0.49)	3.00	207.09	0.68 (2.23)	0
42	Bret	1981/07/01	0.02 (0.07)	1.75	190.00	0.51 (1.67)	0
43	Dean	1983/09/28	0.01 (0.03)	0.70	188.85	0.34 (1.12)	0
44	Gloria	1985/09/27	0.01 (0.03)	3.30	92.45	0.67 (2.20)	0
45	Charley	1986/08/18	0.01 (0.03)	3.30	90.15	0.66 (2.17)	0
46	Danielle	1992/09/26	0.03 (0.10)	1.95	193.45	0.30 (0.98)	0
47	Bertha	1996/07/13	0.18 (0.59)	3.90	189.67	0.79 (2.59)	6
48	Fran	1996/09/07	0.30 (0.98)	4.30	191.15	1.18 (3.87)	24
49	Bonnie	1998/08/28	0.08 (0.26)	3.00	17.00	0.94 (3.08)	0
50	Earl	1998/09/04	0.17 (0.56)	3.25	202.22	0.58 (1.90)	3
51	Floyd	1999/09/16	0.18 (0.59)	3.40	198.56	0.67 (2.20)	9
52	Isabel	2003/09/19	0.35 (1.15)	5.22	182.25	1.83 (6.00)	18

[1]Storm duration is the time during a storm when $H_S > 0.15$ m.

Table D8
Maximum H_s by Storm, James Island, Station 1, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	0.07 (0.23)	5.10	47.90	0.24 (0.79)	0
2	1956/10/17	0.09 (0.30)	3.73	49.00	0.55 (1.80)	0
3	1956/10/28	0.19 (0.62)	3.83	45.96	0.86 (2.82)	12
4	1957/10/05	0.08 (0.26)	3.34	47.95	0.61 (2.00)	0
5	1958/02/15	0.38 (1.25)	3.28	207.73	0.34 (1.12)	15
6	1958/10/21	0.16 (0.52)	3.37	48.04	0.91 (2.99)	6
7	1962/03/08	0.09 (0.30)	3.79	46.90	0.84 (2.76)	0
8	1962/11/27	0.18 (0.59)	3.66	47.00	0.90 (2.95)	18
9	1966/01/30	0.39 (1.28)	3.15	221.67	0.35 (1.15)	30
10	1969/01/21	0.08 (0.26)	3.15	49.00	0.48 (1.57)	0
11	1972/05/24	0.32 (1.05)	2.77	210.27	0.43 (1.41)	6
12	1972/10/07	0.11 (0.36)	2.65	45.96	0.90 (2.95)	0
13	1974/12/02	0.79 (2.59)	4.43	216.36	1.07 (3.51)	48
14	1975/06/28	0.37 (1.21)	2.96	207.73	0.32 (1.05)	6
15	1977/10/29	0.08 (0.26)	3.27	47.95	0.56 (1.84)	0
16	1978/04/27	0.08 (0.26)	3.50	49.00	0.55 (1.80)	0
17	1980/12/29	0.08 (0.26)	3.14	49.00	0.72 (2.36)	0
18	1981/08/20	0.52 (1.71)	3.19	205.00	0.72 (2.36)	9
19	1983/02/11	0.09 (0.30)	3.69	49.00	0.67 (2.20)	0
20	1981/03/29	0.63 (2.07)	4.43	220.75	0.83 (2.72)	9
21	1984/09/26	0.36 (1.18)	3.25	219.13	0.31 (1.02)	3
22	1984/10/14	0.09 (0.30)	3.64	46.90	0.69 (2.26)	0
23	1984/11/19	0.17 (0.56)	2.01	205.20	0.31 (1.02)	3
24	1985/11/05	0.58 (1.90)	3.80	203.00	1.05 (3.44)	30
25	1986/12/02	0.49 (1.61)	3.21	201.77	1.10 (3.61)	18
26	1987/02/17	0.06 (0.20)	3.26	43.92	0.35 (1.15)	0
27	1988/04/13	0.08 (0.26)	3.56	49.00	0.67 (2.20)	0
28	1989/03/08	0.08 (0.26)	3.33	47.95	0.64 (2.10)	0
29	1991/01/08	0.07 (0.23)	2.97	47.95	0.49 (1.61)	0
30	1991/04/19	0.40 (1.31)	2.79	201.27	0.60 (1.97)	9
31	1991/10/31	0.07 (0.23)	2.93	45.85	0.56 (1.84)	0
32	1991/11/10	0.08 (0.26)	3.47	47.95	0.57 (1.87)	0
33	1993/03/14	0.35 (1.15)	3.73	252.99	0.76 (2.49)	27
34	1994/10/15	0.08 (0.26)	3.27	47.95	0.57 (1.87)	0
35	1996/10/09	0.33 (1.08)	2.78	216.60	0.35 (1.15)	3
36	1997/06/06	0.42 (1.38)	2.88	208.73	0.72 (2.36)	18
37	1997/10/14	0.36 (1.18)	2.97	211.53	0.31 (1.02)	12
38	1998/05/14	0.08 (0.26)	2.25	45.96	0.90 (2.95)	0
39	1999/04/29	0.20 (0.66)	2.18	210.27	0.35 (1.15)	3
40	1999/09/05	0.60 (1.97)	3.45	205.00	0.70 (2.30)	45
41	2000/05/30	0.07 (0.23)	3.13	46.90	0.59 (1.94)	0
42	2003/04/11	0.10 (0.33)	2.44	45.96	0.91 (2.99)	0
43	2003/09/09	0.06 (0.02)	2.60	49.00	0.61 (2.00)	0

[1]Storm duration is the time during a storm when H_s > 0.15 m.

Table D9
Maximum H_s by Storm, James Island, Station 3, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	0.56 (1.84)	4.32	284.00	0.15 (0.49)	3
2	1956/10/18	0.35 (1.15)	3.54	294.00	0.55 (1.80)	3
3	1956/10/24	0.15 (0.49)	2.39	294.00	0.31 (1.02)	0
4	1957/10/03	0.23 (0.75)	2.92	292.00	0.22 (0.72)	0
5	1958/02/16	0.78 (2.56)	3.40	282.27	0.55 (1.80)	138
6	1958/10/21	0.11 (0.36)	3.37	349.89	0.91 (2.99)	0
7	1962/03/08	0.65 (2.13)	4.68	287.00	0.70 (2.30)	3
8	1962/12/05	0.56 (1.84)	4.31	286.00	0.46 (1.51)	6
9	1966/01/30	0.99 (3.25)	3.73	257.13	0.18 (0.59)	114
10	1969/01/19	0.11 (0.36)	2.05	294.00	0.21 (0.69)	0
11	1972/05/24	0.49 (1.61)	3.52	211.53	0.26 (0.85)	6
12	1972/10/07	0.52 (1.71)	4.21	287.00	0.73 (2.39)	27
13	1974/12/02	0.95 (3.12)	4.83	217.14	0.96 (3.15)	90
14	1975/06/30	0.83 (2.72)	5.09	284.00	0.16 (0.52)	9
15	1977/10/29	0.07 (0.23)	3.27	319.18	0.56 (1.84)	0
16	1978/04/28	0.60 (1.97)	4.56	287.00	0.57 (1.87)	15
17	1980/12/30	0.52 (1.71)	4.22	286.00	0.42 (1.38)	36
18	1981/08/20	0.45 (1.48)	3.19	198.00	0.72 (2.36)	9
19	1983/02/12	0.55 (1.80)	4.29	286.00	0.51 (1.67)	39
20	1981/03/29	0.88 (2.89)	3.52	244.53	0.99 (3.25)	51
21	1984/10/02	0.45 (1.48)	3.94	296.00	0.59 (1.94)	18
22	1984/10/14	0.08 (0.26)	3.93	318.36	0.57 (1.87)	0
23	1984/11/21	0.53 (1.74)	4.20	284.00	0.13 (0.43)	18
24	1985/11/05	0.66 (2.17)	3.80	195.00	1.05 (3.44)	39
25	1986/12/02	0.48 (1.57)	3.21	193.82	1.10 (3.61)	12
26	1987/02/16	0.23 (0.75)	2.93	292.00	0.21 (0.69)	0
27	1988/04/11	0.12 (0.39)	2.14	294.00	0.31 (1.02)	0
28	1989/03/10	0.38 (1.25)	3.62	296.00	0.57 (1.87)	9
29	1991/01/09	0.22 (0.72)	2.89	296.00	0.61 (2.00)	0
30	1991/04/19	0.35 (1.15)	2.79	194.40	0.60 (1.97)	9
31	1991/10/31	0.37 (1.21)	3.63	294.00	0.35 (1.15)	6
32	1991/11/09	0.28 (0.92)	3.21	294.00	0.19 (0.62)	0
33	1993/03/14	0.98 (3.22)	3.73	278.50	0.76 (2.49)	24
34	1994/10/15	0.07 (0.23)	3.27	319.18	0.57 (1.87)	0
35	1996/10/04	0.41 (1.35)	3.78	294.00	0.34 (1.12)	21
36	1997/06/06	0.37 (1.21)	2.88	201.60	0.72 (2.36)	6
37	1997/10/14	0.37 (1.21)	2.97	204.45	0.31 (1.02)	15
38	1998/05/10	0.17 (0.56)	2.49	294.00	0.31 (1.02)	0
39	1999/04/29	0.24 (0.79)	2.43	208.50	0.13 (0.43)	0
40	1999/09/05	0.53 (1.74)	3.47	195.60	0.67 (2.20)	42
41	2000/05/30	0.06 (0.20)	3.13	318.36	0.59 (1.94)	0
42	2003/04/11	0.42 (1.38)	3.80	296.00	0.66 (2.17)	12
43	2003/09/09	0.05 (0.16)	2.60	320.00	0.61 (2.00)	0

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table D10
Maximum H_s by Storm, James Island, Station 5, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	0.93 (3.05)	4.32	298.00	0.15 (0.49)	42
2	1956/10/18	0.61 (2.00)	3.54	311.00	0.56 (1.84)	33
3	1956/10/28	0.63 (2.07)	4.35	335.14	0.67 (2.20)	96
4	1957/10/05	0.65 (2.13)	4.41	335.14	0.62 (2.03)	42
5	1958/02/16	0.89 (2.92)	4.21	300.00	0.35 (1.15)	132
6	1958/10/20	0.51 (1.67)	3.89	337.13	0.50 (1.64)	54
7	1962/03/08	1.09 (3.58)	4.68	302.00	0.70 (2.30)	69
8	1962/12/05	0.94 (3.08)	4.31	300.00	0.47 (1.54)	183
9	1966/01/27	1.11 (3.64)	4.79	299.00	0.43 (1.41)	132
10	1969/01/21	0.41 (1.35)	3.39	337.00	0.46 (1.51)	51
11	1972/05/26	0.55 (1.80)	3.41	338.87	0.45 (1.48)	54
12	1972/10/07	0.89 (2.92)	4.21	302.00	0.73 (2.39)	36
13	1974/12/04	0.90 (2.95)	4.23	300.00	0.42 (1.38)	105
14	1975/06/30	1.11 (3.64)	5.09	295.00	0.17 (0.56)	51
15	1977/10/30	0.43 (1.41)	3.47	336.14	0.40 (1.31)	48
16	1978/04/28	1.01 (3.31)	4.56	302.00	0.58 (1.90)	48
17	1980/12/30	0.88 (2.89)	4.22	300.00	0.42 (1.38)	90
18	1981/08/20	0.48 (1.57)	2.83	336.14	0.40 (1.31)	36
19	1983/02/12	0.92 (3.02)	4.29	300.00	0.50 (1.64)	78
20	1981/03/30	1.25 (4.10)	5.02	299.00	0.49 (1.61)	69
21	1984/10/02	0.77 (2.53)	3.94	313.00	0.59 (1.94)	141
22	1984/10/14	0.56 (1.84)	4.04	333.28	0.54 (1.77)	57
23	1984/11/21	0.88 (2.89)	4.20	298.00	0.12 (0.39)	63
24	1985/10/28	0.64 (2.10)	3.64	311.00	0.31 (1.02)	129
25	1986/11/30	0.44 (1.44)	3.04	309.00	0.17 (0.56)	60
26	1987/02/16	0.41 (1.35)	2.93	309.00	0.21 (0.69)	30
27	1988/04/13	0.45 (1.48)	3.55	337.00	0.44 (1.44)	36
28	1989/03/10	0.65 (2.13)	3.62	313.00	0.58 (1.90)	84
29	1991/01/09	0.39 (1.28)	2.89	313.00	0.61 (2.00)	42
30	1991/04/21	0.46 (1.51)	3.15	313.00	0.62 (2.03)	6
31	1991/10/31	0.64 (2.10)	3.63	311.00	0.35 (1.15)	39
32	1991/11/09	0.49 (1.61)	3.21	311.00	0.18 (0.59)	54
33	1993/03/14	1.02 (3.35)	3.73	285.94	0.77 (2.53)	48
34	1994/10/15	0.41 (1.35)	3.40	336.14	0.38 (1.25)	18
35	1996/10/04	0.70 (2.30)	3.78	311.00	0.34 (1.12)	63
36	1997/06/04	0.37 (1.21)	3.14	337.00	0.37 (1.21)	27
37	1997/10/19	0.60 (1.97)	3.52	311.00	0.42 (1.38)	78
38	1998/05/12	0.33 (1.08)	3.13	337.13	0.60 (1.97)	18
39	1999/05/03	0.41 (1.35)	2.95	311.00	0.42 (1.38)	24
40	1999/08/30	0.48 (1.57)	3.70	335.27	0.14 (0.46)	126
41	2000/05/29	0.45 (1.48)	3.57	336.14	0.38 (1.25)	27
42	2003/04/12	0.73 (2.39)	3.80	313.00	0.65 (2.13)	51
43	2003/09/10	0.37 (1.21)	3.20	336.14	0.34 (1.12)	15

[1]Storm duration is the time during a storm when $H_s > 0.3$ m.

Table D11
Maximum H_s by Storm, James Island, Station 7, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	1.04 (3.41)	4.32	313.00	0.15 (0.49)	45
2	1956/10/18	0.66 (2.17)	3.54	319.00	0.56 (1.84)	36
3	1956/10/25	0.71 (2.33)	4.24	341.12	0.22 (0.72)	99
4	1957/10/05	0.73 (2.39)	4.41	343.12	0.62 (2.03)	48
5	1958/02/16	0.97 (3.18)	4.21	314.00	0.35 (1.15)	129
6	1958/10/20	0.58 (1.90)	3.89	349.10	0.50 (1.64)	60
7	1962/03/08	1.17 (3.84)	4.68	314.00	0.70 (2.30)	69
8	1962/12/05	1.03 (3.38)	4.31	314.00	0.47 (1.54)	201
9	1966/01/27	1.23 (4.04)	4.79	314.00	0.43 (1.41)	126
10	1969/01/21	0.44 (1.44)	3.39	348.00	0.46 (1.51)	57
11	1972/05/26	0.61 (2.00)	3.41	350.90	0.45 (1.48)	60
12	1972/10/07	0.95 (3.12)	4.21	314.00	0.73 (2.39)	42
13	1974/12/04	1.00 (3.28)	4.24	313.00	0.16 (0.52)	96
14	1975/06/30	1.28 (4.20)	5.09	312.00	0.17 (0.56)	51
15	1977/10/30	0.47 (1.54)	3.47	347.11	0.40 (1.31)	69
16	1978/04/28	1.08 (3.54)	4.56	314.00	0.58 (1.90)	48
17	1980/12/30	0.97 (3.18)	4.22	314.00	0.42 (1.38)	102
18	1981/08/21	0.51 (1.67)	3.60	347.11	0.40 (1.31)	30
19	1983/02/12	1.01 (3.31)	4.29	314.00	0.50 (1.64)	81
20	1981/03/30	1.39 (4.56)	5.02	314.00	0.49 (1.61)	69
21	1984/10/02	0.82 (2.69)	3.94	319.00	0.59 (1.94)	141
22	1984/10/14	0.63 (2.07)	4.04	341.24	0.54 (1.77)	57
23	1984/11/21	0.98 (3.22)	4.20	313.00	0.12 (0.39)	66
24	1985/10/28	0.70 (2.30)	3.64	319.00	0.31 (1.02)	129
25	1986/12/01	0.49 (1.61)	3.45	346.11	0.14 (0.46)	60
26	1987/02/16	0.44 (1.44)	2.93	319.00	0.21 (0.69)	30
27	1988/04/13	0.49 (1.61)	3.55	348.00	0.44 (1.44)	39
28	1989/03/10	0.69 (2.26)	3.62	319.00	0.58 (1.90)	84
29	1991/01/07	0.42 (1.38)	3.20	346.11	0.09 (0.30)	45
30	1991/04/21	0.50 (1.64)	3.13	319.00	0.46 (1.51)	6
31	1991/10/31	0.70 (2.30)	3.63	319.00	0.35 (1.15)	48
32	1991/11/09	0.53 (1.74)	3.21	319.00	0.18 (0.59)	54
33	1993/03/14	1.04 (3.41)	3.73	292.31	0.77 (2.53)	51
34	1994/10/15	0.44 (1.44)	3.40	347.11	0.38 (1.25)	18
35	1996/10/04	0.77 (2.53)	3.78	319.00	0.34 (1.12)	72
36	1997/06/04	0.39 (1.28)	3.14	348.00	0.37 (1.21)	24
37	1997/10/19	0.66 (2.17)	3.52	319.00	0.42 (1.38)	84
38	1998/05/12	0.35 (1.15)	3.13	349.10	0.60 (1.97)	21
39	1999/05/03	0.45 (1.48)	2.95	319.00	0.42 (1.38)	27
40	1999/08/30	0.54 (1.77)	3.70	345.22	0.14 (0.46)	96
41	2000/05/29	0.49 (1.61)	3.57	347.11	0.38 (1.25)	27
42	2003/04/12	0.77 (2.53)	3.80	319.00	0.65 (2.13)	51
43	2003/09/10	0.40 (1.31)	3.20	347.11	0.34 (1.12)	15

[1]Storm duration is the time during a storm when H_S > 0.3 m.

Table D12
Maximum H_s by Storm, James Island, Station 9, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	0.86 (2.82)	4.32	332.00	0.12 (0.39)	51
2	1956/10/17	0.65 (2.13)	3.73	20.00	0.60 (1.97)	51
3	1956/10/28	0.74 (2.43)	4.35	12.04	0.69 (2.26)	147
4	1957/10/05	0.75 (2.46)	4.41	12.04	0.64 (2.10)	54
5	1958/02/16	0.81 (2.66)	4.21	330.00	0.38 (1.25)	126
6	1958/10/21	0.67 (2.20)	3.80	19.03	0.62 (2.03)	72
7	1962/03/08	1.01 (3.31)	4.68	329.00	0.68 (2.23)	75
8	1962/12/05	0.86 (2.82)	4.31	330.00	0.49 (1.61)	228
9	1966/01/27	1.06 (3.48)	4.79	330.00	0.46 (1.51)	132
10	1969/01/21	0.53 (1.74)	3.39	16.00	0.43 (1.41)	63
11	1972/05/26	0.66 (2.17)	3.41	11.95	0.44 (1.44)	75
12	1972/10/07	0.81 (2.66)	4.21	329.00	0.73 (2.39)	105
13	1974/12/04	0.83 (2.72)	4.23	330.00	0.45 (1.48)	66
14	1975/06/30	1.22 (4.00)	5.09	331.00	0.18 (0.59)	66
15	1977/10/30	0.55 (1.80)	3.47	15.04	0.43 (1.41)	117
16	1978/04/28	0.93 (3.05)	4.56	329.00	0.61 (2.00)	51
17	1980/12/30	0.81 (2.66)	4.22	330.00	0.44 (1.44)	111
18	1981/08/20	0.65 (2.13)	2.83	15.04	0.37 (1.21)	57
19	1983/02/12	0.85 (2.79)	4.29	330.00	0.49 (1.61)	120
20	1981/03/30	1.22 (4.00)	5.23	328.00	0.62 (2.03)	66
21	1984/10/02	0.69 (2.26)	3.94	323.00	0.62 (2.03)	138
22	1984/10/14	0.64 (2.10)	3.93	11.09	0.57 (1.87)	99
23	1984/11/21	0.80 (2.62)	4.20	332.00	0.12 (0.39)	78
24	1985/11/05	0.64 (2.10)	3.25	78.15	1.13 (3.71)	153
25	1986/12/02	0.59 (1.94)	3.41	20.00	0.72 (2.36)	69
26	1987/02/17	0.51 (1.67)	3.26	14.07	0.38 (1.25)	48
27	1988/04/13	0.60 (1.97)	3.56	20.00	0.66 (2.17)	60
28	1989/03/10	0.58 (1.90)	3.62	323.00	0.60 (1.97)	96
29	1991/01/08	0.48 (1.57)	2.97	19.03	0.48 (1.57)	78
30	1991/04/21	0.41 (1.35)	2.73	19.03	0.62 (2.03)	24
31	1991/10/31	0.58 (1.90)	3.63	325.00	0.36 (1.18)	66
32	1991/11/10	0.58 (1.90)	3.47	19.03	0.57 (1.87)	51
33	1993/03/14	0.70 (2.30)	3.73	323.14	0.80 (2.62)	51
34	1994/10/15	0.54 (1.77)	3.27	19.03	0.57 (1.87)	90
35	1996/10/04	0.63 (2.07)	3.78	325.00	0.35 (1.15)	144
36	1997/06/04	0.49 (1.61)	3.14	16.00	0.35 (1.15)	90
37	1997/10/19	0.54 (1.77)	3.52	325.00	0.39 (1.28)	129
38	1998/05/12	0.50 (1.64)	3.13	19.03	0.58 (1.90)	69
39	1999/05/02	0.51 (1.67)	3.27	15.04	0.55 (1.80)	81
40	1999/08/30	0.59 (1.94)	3.60	15.04	0.32 (1.05)	144
41	2000/05/30	0.57 (1.87)	3.56	16.00	0.39 (1.28)	54
42	2003/04/12	0.65 (2.13)	3.80	323.00	0.64 (2.10)	96
43	2003/09/10	0.49 (1.61)	3.20	15.04	0.34 (1.12)	66

[1]Storm duration is the time during a storm when $H_s > 0.3$ m.

Table D13
Maximum H_s by Storm, James Island, Station 11, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	0.60 (1.97)	5.10	23.03	0.30 (0.98)	54
2	1956/10/17	0.49 (1.61)	3.73	25.00	0.60 (1.97)	69
3	1956/10/28	0.75 (2.46)	3.83	34.98	0.90 (2.95)	153
4	1957/10/05	0.53 (1.74)	4.41	25.01	0.64 (2.10)	66
5	1958/02/16	0.31 (1.02)	4.21	341.00	0.38 (1.25)	123
6	1958/10/21	0.62 (2.03)	3.37	37.02	0.91 (2.99)	72
7	1962/03/07	0.53 (1.74)	4.41	24.02	0.75 (2.46)	84
8	1962/11/27	0.71 (2.33)	3.66	36.00	0.88 (2.89)	228
9	1966/01/27	0.47 (1.54)	3.82	22.02	0.26 (0.85)	132
10	1969/01/21	0.38 (1.25)	3.15	25.00	0.43 (1.41)	66
11	1972/05/27	0.60 (1.97)	3.24	36.00	0.94 (3.08)	87
12	1972/10/07	0.43 (1.41)	2.65	34.98	0.90 (2.95)	108
13	1974/12/02	0.70 (2.30)	3.41	91.85	0.95 (3.12)	60
14	1975/06/30	0.43 (1.41)	3.59	21.04	0.33 (1.08)	75
15	1977/10/29	0.40 (1.31)	3.27	24.01	0.58 (1.90)	120
16	1978/04/27	0.45 (1.48)	3.50	25.00	0.60 (1.97)	60
17	1980/12/29	0.38 (1.25)	3.14	25.00	0.72 (2.36)	117
18	1981/08/20	0.47 (1.54)	3.66	24.01	0.58 (1.90)	93
19	1983/02/11	0.48 (1.57)	3.69	25.00	0.71 (2.33)	120
20	1981/03/29	0.47 (1.54)	4.11	26.99	0.71 (2.33)	66
21	1984/09/30	0.47 (1.54)	3.63	24.01	0.80 (2.62)	135
22	1984/10/14	0.47 (1.54)	3.64	23.03	0.69 (2.26)	108
23	1984/11/20	0.38 (1.25)	3.31	21.04	0.36 (1.18)	75
24	1985/11/05	0.65 (2.13)	3.25	91.85	1.13 (3.71)	177
25	1986/12/02	0.48 (1.57)	2.73	91.85	1.02 (3.35)	72
26	1987/02/17	0.37 (1.21)	3.26	21.04	0.38 (1.25)	48
27	1988/04/13	0.45 (1.48)	3.56	25.00	0.66 (2.17)	63
28	1989/03/08	0.42 (1.38)	3.33	24.01	0.62 (2.03)	96
29	1991/01/08	0.35 (1.15)	2.97	24.01	0.48 (1.57)	78
30	1991/04/21	0.30 (0.98)	2.73	24.01	0.62 (2.03)	66
31	1991/10/31	0.35 (1.15)	3.15	20.05	0.53 (1.74)	54
32	1991/11/10	0.43 (1.41)	3.47	24.01	0.57 (1.87)	45
33	1993/03/13	0.46 (1.51)	3.81	23.00	0.43 (1.41)	39
34	1994/10/15	0.40 (1.31)	3.27	24.01	0.57 (1.87)	108
35	1996/10/08	0.43 (1.41)	3.42	26.97	0.77 (2.53)	141
36	1997/06/04	0.35 (1.15)	3.14	23.00	0.35 (1.15)	105
37	1997/10/16	0.37 (1.21)	3.26	21.04	0.47 (1.54)	117
38	1998/05/12	0.37 (1.21)	3.13	24.01	0.58 (1.90)	90
39	1999/05/02	0.37 (1.21)	3.27	22.02	0.55 (1.80)	93
40	1999/08/30	0.43 (1.41)	3.60	22.02	0.32 (1.05)	144
41	2000/05/29	0.41 (1.35)	3.57	22.02	0.35 (1.15)	66
42	2003/04/10	0.48 (1.57)	3.75	25.99	0.57 (1.87)	105
43	2003/09/10	0.35 (1.15)	3.20	22.02	0.34 (1.12)	96

[1]Storm duration is the time during a storm when $H_S > 0.15$ m.

Table D14
Maximum H_s by Storm, James Island, Station 13, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw	Duration, hr[1]
1	1945/01/23	0.01 (0.03)	2.36	86.70	0.53 (1.74)	0
2	1956/10/17	0.01 (0.03)	3.35	90.15	0.69 (2.26)	0
3	1956/10/28	0.11 (0.36)	3.83	16.03	0.92 (3.02)	0
4	1957/10/05	0.01 (0.03)	3.34	87.85	0.62 (2.03)	0
5	1958/02/15	0.08 (0.26)	3.28	188.85	0.35 (1.15)	0
6	1958/10/21	0.09 (0.30)	3.37	17.97	0.92 (3.02)	0
7	1962/03/06	0.01 (0.03)	3.05	87.85	0.80 (2.62)	0
8	1962/11/27	0.10 (0.33)	3.66	17.00	0.89 (2.92)	0
9	1966/01/30	0.07 (0.23)	3.15	201.52	0.35 (1.15)	0
10	1969/01/21	0.01 (0.03)	3.15	89.00	0.43 (1.41)	0
11	1972/05/27	0.09 (0.30)	3.24	17.00	0.95 (3.12)	0
12	1972/10/07	0.06 (0.20)	2.65	16.03	0.90 (2.95)	0
13	1974/12/02	0.23 (0.75)	4.43	210.18	1.09 (3.58)	21
14	1975/06/28	0.07 (0.23)	2.96	188.85	0.32 (1.05)	0
15	1977/11/01	0.01 (0.03)	2.99	90.15	0.58 (1.90)	0
16	1978/04/26	0.01 (0.03)	3.40	89.00	0.68 (2.23)	0
17	1980/12/29	0.01 (0.03)	3.14	89.00	0.72 (2.36)	0
18	1981/08/20	0.16 (0.52)	3.19	201.00	0.71 (2.33)	3
19	1983/02/15	0.01 (0.03)	3.05	86.70	0.62 (2.03)	0
20	1981/03/29	0.15 (0.49)	4.43	212.45	0.81 (2.66)	0
21	1984/09/26	0.08 (0.26)	3.25	199.21	0.31 (1.02)	0
22	1984/10/14	0.01 (0.03)	3.64	86.70	0.70 (2.30)	0
23	1984/11/19	0.03 (0.10)	2.01	186.55	0.31 (1.02)	0
24	1985/11/05	0.23 (0.75)	3.35	173.10	0.96 (3.15)	12
25	1986/12/02	0.21 (0.69)	3.21	167.98	1.12 (3.67)	6
26	1987/02/15	0.01 (0.03)	2.02	81.86	0.31 (1.02)	0
27	1988/04/13	0.01 (0.03)	3.24	90.15	0.62 (2.03)	0
28	1989/03/09	0.01 (0.03)	3.14	85.55	0.80 (2.62)	0
29	1991/01/08	0.01 (0.03)	2.97	87.85	0.49 (1.61)	0
30	1991/04/19	0.13 (0.43)	2.79	197.35	0.58 (1.90)	0
31	1991/10/31	0.01 (0.03)	2.93	85.55	0.61 (2.00)	0
32	1991/11/09	0.01 (0.03)	3.12	86.70	0.56 (1.84)	0
33	1993/03/14	0.07 (0.23)	3.42	0.00	0.94 (3.08)	0
34	1994/10/15	0.01 (0.03)	3.27	87.85	0.58 (1.90)	0
35	1996/10/09	0.06 (0.20)	2.78	196.91	0.35 (1.15)	0
36	1997/06/06	0.13 (0.43)	2.88	204.65	0.71 (2.33)	0
37	1997/10/14	0.07 (0.23)	2.97	192.30	0.31 (1.02)	0
38	1998/05/14	0.05 (0.16)	2.25	16.03	0.94 (3.08)	0
39	1999/04/29	0.03 (0.10)	2.18	191.15	0.31 (1.02)	0
40	1999/09/05	0.19 (0.62)	3.47	198.56	0.70 (2.30)	12
41	2000/05/31	0.01 (0.03)	1.08	204.65	0.80 (2.62)	0
42	2003/04/11	0.06 (0.20)	2.44	16.03	0.94 (3.08)	0
43	2003/09/09	0.01 (0.03)	2.60	89.00	0.65 (2.13)	0

[1]Storm duration is the time during a storm when H_s > 0.15 m.

Appendix E
Extremal Wave and Water Level Analysis Results for James Island

Extremal analysis of significant wave heights was applied to all storms together and to hurricanes only, and results are summarized in this appendix. Analysis of all storms included 179 storms over the 148-year time period. Analysis of hurricanes only included 52 storms over the 148-year period. The best-fitting extremal distribution was selected, based on the criteria of Goda and Kobune (1990) and a good visual fit to the return periods of concern for this project. Using the best-fit distribution, significant wave heights were determined for return periods of 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, and 100 years. For hurricane-influenced stations where the best-fit distribution for all storms underestimated H_s at the longest return periods, return period H_s was taken from the best fit for hurricanes only for return periods dominated by hurricanes.

To estimate an appropriate peak wave period and water level to accompany each return-period significant wave height, the computer program return_period_Tp.f is run. Inputs include return-period significant wave heights and 148-year time history of waves and water levels at each station. The time history is screened to find all significant heights within a bin centered on the desired return period wave height. Bin widths considered are 0.2, 0.4, 0.6, 0.8, and 1.0 m (0.7, 1.3, 2.0, 2.6, and 3.3 ft). For each return period, a representative or *average* period and water level were chosen with consideration of bins that captured enough cases to form a meaningful average but not so many cases as to dilute the target severe events.

Tables E1-E13 summarize extremal wave height analysis results for sta 1-13 of James Island. Tables E14 and E15 give results of an extremal analysis of maximum storm water levels for James Island. The maximum water levels for each storm were fit to a Fisher-Tippett type I distribution. The extremal water levels, referenced to msl, associated with northeasters from Table 17 in Chapter 3 are listed in Table E14. The extremal water levels, referenced to msl, associated with tropical storms from Table 14 in Chapter 3 are listed in Table E15. The relationship used here for James island tidal datums is msl = 0.208 m mllw. As described earlier for Poplar Island, these extremal water levels are only for storms. The extremal analysis did not include all water levels throughout the year (e.g., spring tide).

Table E1
James Island Station 1 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.53 (1.74)	3.32	0.83 (2.72)
10	0.68 (2.23)	4.44	1.03 (3.38)
15	0.76 (2.49)	4.68	1.14 (3.74)
20	0.82 (2.69)	5.16	1.29 (4.23)
25	0.86 (2.82)	5.60	1.35 (4.43)
30	0.90 (2.95)	5.88	1.40 (4.59)
35	0.93 (3.05)	6.02	1.41 (4.63)
40	0.95 (3.12)	6.58	1.47 (4.82)
45	0.97 (3.18)	6.55	1.47 (4.82)
50	0.99 (3.25)	6.79	1.59 (5.22)
100	1.11 (3.64)	8.64	1.67 (5.48)

Table E2
James Island Station 2 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.66 (2.17)	3.31	0.80 (2.62)
10	0.92 (3.02)	4.38	0.94 (3.08)
15	1.07 (3.51)	4.77	1.06 (3.48)
20	1.18 (3.87)	5.29	1.09 (3.58)
25	1.26 (4.13)	5.52	1.19 (3.90)
30	1.33 (4.36)	5.63	1.37 (4.49)
35	1.38 (4.53)	6.26	1.50 (4.92)
40	1.43 (4.69)	6.55	1.42 (4.66)
45	1.48 (4.86)	6.74	1.43 (4.69)
50	1.52 (4.99)	6.79	1.48 (4.86)
100	1.77 (5.81)	8.64	1.57 (5.15)

Table E3
James Island Station 3 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.79 (2.59)	3.79	0.52 (1.71)
10	1.05 (3.44)	4.52	0.70 (2.30)
15	1.20 (3.94)	5.00	1.36 (4.46)
20	1.30 (4.27)	5.42	1.39 (4.56)
25	1.38 (4.53)	5.71	1.27 (4.17)
30	1.45 (4.76)	5.60	1.36 (4.46)
35	1.50 (4.92)	5.86	1.27 (4.17)
40	1.55 (5.09)	5.93	1.33 (4.36)
45	1.60 (5.25)	5.98	1.41 (4.63)
50	1.64 (5.38)	6.12	1.26 (4.13)
100	1.89 (6.20)	7.94	1.32 (4.33)

Table E4
James Island Station 4 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.91 (2.99)	4.05	0.49 (1.61)
10	1.14 (3.74)	4.97	0.94 (3.08)
15	1.28 (4.20)	5.23	1.10 (3.61)
20	1.37 (4.49)	5.53	1.34 (4.40)
25	1.45 (4.76)	5.75	1.37 (4.49)
30	1.51 (4.95)	5.98	1.30 (4.27)
35	1.56 (5.12)	6.21	1.33 (4.36)
40	1.61 (5.28)	6.58	1.37 (4.49)
45	1.64 (5.38)	6.69	1.11 (3.64)
50	1.68 (5.51)	6.53	1.11 (3.64)
100	1.91 (6.27)	7.60	1.35 (4.43)

Table E5 James Island Station 5 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.98 (3.22)	4.99	0.53 (1.74)
10	1.16 (3.81)	5.13	0.69 (2.26)
15	1.26 (4.13)	5.48	1.04 (3.41)
20	1.34 (4.40)	5.97	1.08 (3.54)
25	1.39 (4.56)	6.11	1.34 (4.40)
30	1.44 (4.72)	6.01	1.27 (4.17)
35	1.48 (4.86)	6.05	1.33 (4.36)
40	1.51 (4.95)	6.12	1.37 (4.49)
45	1.55 (5.09)	6.22	1.11 (3.64)
50	1.59 (5.22)	6.22	1.32 (4.33)
100	1.89 (6.20)	7.55	1.35 (4.43)

Table E6 James Island Station 6 Extremal Wave Analysis Results			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	1.10 (3.61)	4.64	0.48 (1.57)
10	1.25 (4.10)	5.48	0.65 (2.13)
15	1.34 (4.40)	5.76	0.78 (2.56)
20	1.39 (4.56)	6.03	0.88 (2.89)
25	1.43 (4.69)	6.03	0.77 (2.53)
30	1.46 (4.79)	6.07	0.73 (2.40)
35	1.49 (4.89)	6.12	0.90 (2.95)
40	1.51 (4.95)	6.22	0.82 (2.69)
45	1.54 (5.05)	6.22	0.82 (2.69)
50	1.58 (5.18)	6.71	0.81 (2.66)
100	1.86 (6.10)	7.81	1.42 (4.66)

Table E7
James Island Station 7 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	1.03 (3.38)	4.98	0.45 (1.48)
10	1.19 (3.90)	5.58	0.57 (1.87)
15	1.28 (4.20)	5.78	0.57 (1.87)
20	1.33 (4.36)	5.76	0.59 (1.94)
25	1.37 (4.49)	5.79	0.59 (1.94)
30	1.41 (4.63)	6.03	0.69 (2.26)
35	1.43 (4.69)	6.35	0.69 (2.26)
40	1.46 (4.79)	6.55	0.69 (2.26)
45	1.48 (4.86)	6.77	0.68 (2.23)
50	1.49 (4.89)	6.77	0.75 (2.46)
100	1.71 (5.61)	7.94	1.35 (4.43)

Table E8
James Island Station 8 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	1.01 (3.31)	4.98	0.39 (1.28)
10	1.16 (3.81)	5.55	0.45 (1.48)
15	1.24 (4.07)	5.78	0.60 (1.97)
20	1.30 (4.27)	5.64	0.60 (1.97)
25	1.34 (4.40)	5.94	0.67 (2.20)
30	1.38 (4.53)	6.28	0.62 (2.03)
35	1.41 (4.63)	6.35	0.54 (1.77)
40	1.43 (4.69)	6.49	0.64 (2.10)
45	1.46 (4.79)	6.77	0.67 (2.20)
50	1.48 (4.86)	6.77	0.62 (2.03)
100	1.62 (5.31)	7.94	0.54 (1.77)

Table E9
James Island Station 9 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.83 (2.72)	4.93	0.39 (1.28)
10	0.96 (3.15)	5.55	0.41 (1.35)
15	1.03 (3.38)	5.89	0.54 (1.77)
20	1.08 (3.54)	5.77	0.53 (1.74)
25	1.12 (3.67)	5.99	0.52 (1.71)
30	1.15 (3.77)	6.36	0.53 (1.74)
35	1.18 (3.87)	6.45	0.56 (1.84)
40	1.20 (3.94)	6.77	0.56 (1.84)
45	1.22 (4.00)	6.77	0.53 (1.74)
50	1.24 (4.07)	6.84	0.53 (1.74)
100	1.36 (4.46)	7.94	0.47 (1.54)

Table E10
James Island Station 10 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.68 (2.23)	5.14	0.62 (2.03)
10	0.74 (2.43)	5.81	0.67 (2.20)
15	0.78 (2.56)	5.99	0.70 (2.30)
20	0.80 (2.62)	6.27	0.69 (2.26)
25	0.82 (2.69)	6.42	0.68 (2.23)
30	0.83 (2.72)	6.28	0.67 (2.20)
35	0.84 (2.76)	6.35	0.66 (2.17)
40	0.85 (2.79)	6.35	0.66 (2.17)
45	0.86 (2.82)	6.35	0.66 (2.17)
50	0.87 (2.85)	6.35	0.66 (2.17)
100	0.91 (2.99)	8.46	0.60 (1.97)

Table E11
James Island Station 11 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.58 (1.90)	5.18	0.79 (2.59)
10	0.65 (2.13)	5.79	0.92 (3.02)
15	0.69 (2.26)	6.06	0.93 (3.05)
20	0.71 (2.33)	6.36	0.96 (3.15)
25	0.73 (2.40)	6.38	0.96 (3.15)
30	0.75 (2.46)	6.43	0.96 (3.15)
35	0.76 (2.49)	6.35	0.95 (3.12)
40	0.77 (2.53)	6.26	0.95 (3.12)
45	0.78 (2.56)	6.82	0.95 (3.12)
50	0.79 (2.59)	6.82	0.94 (3.08)
100	0.83 (2.72)	8.46	0.98 (3.22)

Table E12
James Island Station 12 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.36 (1.18)	4.51	0.59 (1.94)
10	0.39 (1.28)	5.45	0.63 (2.07)
15	0.41 (1.35)	5.67	0.66 (2.17)
20	0.42 (1.38)	6.05	0.72 (2.36)
25	0.43 (1.41)	6.14	0.72 (2.36)
30	0.44 (1.44)	6.17	0.75 (2.46)
35	0.44 (1.44)	6.17	0.81 (2.66)
40	0.45 (1.48)	6.44	0.81 (2.66)
45	0.45 (1.48)	6.64	0.85 (2.79)
50	0.45 (1.48)	7.36	0.85 (2.79)
100	0.48 (1.57)	7.65	0.89 (2.92)

Table E13
James Island Station 13 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.17 (0.56)	4.31	0.80 (2.62)
10	0.22 (0.72)	4.84	0.89 (2.92)
15	0.25 (0.82)	5.37	0.99 (3.25)
20	0.26 (0.85)	5.67	1.05 (3.44)
25	0.28 (0.92)	5.75	1.10 (3.61)
30	0.29 (0.95)	5.98	1.21 (3.97)
35	0.30 (0.98)	6.09	1.24 (4.07)
40	0.31 (1.02)	6.16	1.24 (4.07)
45	0.32 (1.05)	7.02	1.31 (4.30)
50	0.32 (1.05)	7.73	1.33 (4.36)
100	0.37 (1.21)	7.65	1.48 (4.86)

Table E14
Extreme Water Levels for Historical Northeasters from James Island Water Level Analysis Station 3 (Figure 18)

Return Period in Years	Water Level Relative to mllw (ft)
5	2.26
10	2.62
25	3.07
50	3.4
100	3.73

Table E15
Extreme Water Levels for Historical Hurricanes from James Island Water Level Analysis Station 3 (Figure 18)

Return Period in Years	Water Level Relative to mllw (ft)
5	2.01
10	2.82
25	3.86
50	4.62
100	5.38

Appendix F
Armor Weight as Function of Return Period for James Island

This appendix summarizes stable main armor, filter, and toe stone weight and nominal diameter as a function of return period for each design analysis station of James Island. The armor stone sizes were computed using Equations 30-34 and the extremal waves from Appendix E. The results for a seaward structure slope of cot α = 3.0 are shown. Armor and filter layer thicknesses can be computed as $2D_{50}$. For conversion to metric, use 0.3048 m = 1 ft and 4.44822 N = 1 lb.

Table F1
Armor Weight as Function of Return Period for Station 1 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	0.778	78	0.361	8	1.677	782
10	1.024	178	0.476	18	1.706	827
15	1.145	249	0.531	25	1.719	847
20	1.237	316	0.574	31	0.846	100
25	1.293	358	0.600	35	0.873	111
30	1.375	433	0.640	43	0.899	120
35	1.414	471	0.656	47	0.919	129
40	1.444	500	0.669	50	0.932	136
45	1.483	542	0.689	54	0.945	140
50	1.529	596	0.712	59	0.958	147
100	1.677	784	0.778	78	1.040	187

Table F2
Armor Weight as Function of Return Period for Station 2 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	0.965	149	0.449	15	2.126	1600
10	1.345	404	0.623	40	2.352	2162
15	1.568	642	0.728	64	2.313	2053
20	1.745	884	0.810	88	2.303	2033
25	1.854	1062	0.863	105	2.290	1996
30	1.946	1224	0.902	121	2.274	1956
35	2.034	1402	0.945	139	2.280	1973
40	2.123	1591	0.988	158	2.293	2007
45	2.195	1760	1.020	174	2.297	2013
50	2.277	1964	1.056	195	2.310	2047
100	2.592	2893	1.204	287	2.303	2036

Table F3
Armor Weight as Function of Return Period for Station 3 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.161	260	0.538	26	2.198	1764
10	1.486	547	0.689	54	2.352	2167
15	1.785	949	0.830	94	2.497	2591
20	1.936	1207	0.899	120	2.559	2789
25	2.064	1462	0.958	145	2.618	2982
30	2.169	1696	1.007	168	2.654	3116
35	2.231	1849	1.037	183	2.641	3060
40	2.300	2027	1.070	201	2.625	3009
45	2.369	2216	1.102	220	2.612	2964
50	2.418	2356	1.122	233	2.605	2944
100	2.808	3682	1.306	365	2.602	2936

Table F4 Armor Weight as Function of Return Period for Station 4 at James Island						
Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.316	380	0.610	38	2.277	1964
10	1.680	791	0.781	78	2.454	2460
15	1.880	1104	0.873	109	2.536	2713
20	2.067	1473	0.961	146	2.618	2987
25	2.172	1707	1.010	169	2.654	3116
30	2.247	1891	1.043	187	2.661	3133
35	2.316	2069	1.076	205	2.644	3080
40	2.388	2271	1.109	225	2.635	3044
45	2.428	2378	1.129	236	2.635	3044
50	2.474	2520	1.148	250	2.625	3007
100	2.838	3798	1.319	376	2.618	2987

Table F5 Armor Weight as Function of Return Period for Station 5 at James Island						
Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.414	471	0.656	47	2.316	2064
10	1.690	804	0.784	80	2.448	2442
15	1.870	1089	0.869	108	2.526	2684
20	2.001	1331	0.928	132	2.582	2860
25	2.123	1591	0.988	158	2.631	3036
30	2.195	1758	1.020	174	2.661	3136
35	2.234	1853	1.037	184	2.671	3169
40	2.277	1967	1.060	195	2.687	3224
45	2.333	2113	1.083	209	2.713	3324
50	2.369	2216	1.102	220	2.717	3333
100	2.831	3769	1.316	374	2.736	3411

Table F6
Armor Weight as Function of Return Period for Station 6 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.555	624	0.722	62	2.382	2247
10	1.804	978	0.837	97	2.500	2600
15	1.929	1196	0.896	119	2.549	2756
20	1.998	1324	0.928	131	2.572	2833
25	2.044	1422	0.948	141	2.592	2900
30	2.106	1553	0.978	154	2.621	3002
35	2.146	1642	0.997	163	2.631	3033
40	2.156	1669	1.001	165	2.635	3042
45	2.192	1753	1.017	174	2.648	3082
50	2.270	1944	1.053	193	2.680	3204
100	2.818	3729	1.309	370	2.703	3291

Table F7
Armor Weight as Function of Return Period for Station 7 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.470	529	0.682	52	2.343	2136
10	1.722	849	0.801	84	2.461	2484
15	1.834	1029	0.853	102	2.507	2624
20	1.909	1158	0.886	115	2.539	2727
25	1.969	1267	0.915	126	2.566	2804
30	2.057	1451	0.955	144	2.605	2944
35	2.083	1502	0.968	149	2.615	2973
40	2.116	1580	0.984	157	2.628	3016
45	2.116	1576	0.981	156	2.618	2987
50	2.126	1596	0.988	158	2.618	2987
100	2.612	2962	1.214	294	2.782	3584

Table F8
Armor Weight as Function of Return Period for Station 8 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, i lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.450	507	0.673	50	2.316	2069
10	1.667	771	0.774	76	2.418	2356
15	1.795	962	0.833	95	2.477	2524
20	1.886	1113	0.876	110	2.516	2647
25	1.969	1267	0.915	126	2.552	2769
30	2.028	1387	0.942	137	2.579	2853
35	2.041	1416	0.948	140	2.579	2856
40	2.064	1460	0.958	145	2.585	2869
45	2.110	1562	0.981	155	2.602	2933
50	2.146	1644	0.997	163	2.618	2989
100	2.287	1993	1.063	198	2.661	3131

Table F9
Armor Weight as Function of Return Period for Station 9 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	1.243	320	0.577	32	2.221	1820
10	1.430	487	0.663	48	2.316	2067
15	1.532	600	0.712	59	2.365	2202
20	1.594	676	0.741	67	2.392	2278
25	1.650	747	0.768	74	2.418	2349
30	1.696	813	0.787	81	2.438	2416
35	1.742	880	0.810	87	2.457	2471
40	1.765	916	0.820	91	2.467	2500
45	1.781	940	0.827	93	2.470	2511
50	1.804	978	0.840	97	2.480	2538
100	1.959	1249	0.909	124	2.539	2729

Table F10 Armor Weight as Function of Return Period for Station 10 at James Island						
Return Period, years	**Armor Nominal Diameter D_{n50}, ft**	**Armor Weight $W_{a,50}$, lb**	**Filter Layer Nominal Diameter D_{u50}, ft**	**Filter Layer Weight W_{u50}, lb**	**Toe Armor Nominal Diameter $D_{toe,50}$, ft**	**Toe Armor Weight $W_{toe,50}$, ft**
5	1.030	182	0.479	18	2.093	1527
10	1.129	240	0.525	24	2.156	1669
15	1.194	282	0.554	28	2.192	1756
20	1.227	307	0.571	30	2.215	1802
25	1.253	329	0.584	33	2.228	1840
30	1.289	356	0.597	35	2.251	1896
35	1.309	373	0.607	37	2.264	1927
40	1.322	384	0.614	38	2.267	1942
45	1.335	396	0.620	39	2.274	1958
50	1.345	407	0.627	40	2.280	1973
100	1.411	467	0.656	46	2.313	2062

Table F11 Armor Weight as Function of Return Period for Station 11 at James Island						
Return Period, years	**Armor Nominal Diameter D_{n50}, ft**	**Armor Weight $W_{a,50}$, lb**	**Filter Layer Nominal Diameter D_{u50}, ft**	**Filter Layer Weight W_{u50}, lb**	**Toe Armor Nominal Diameter $D_{toe,50}$, ft**	**Toe Armor Weight $W_{toe,50}$, ft**
5	0.879	113	0.410	11	2.018	1367
10	0.958	147	0.443	15	2.060	1458
15	1.007	169	0.469	17	2.093	1522
20	1.027	180	0.476	18	2.103	1544
25	1.056	198	0.492	20	2.123	1589
30	1.083	211	0.502	21	2.136	1622
35	1.102	224	0.512	22	2.149	1653
40	1.115	231	0.518	23	2.156	1669
45	1.129	240	0.525	24	2.162	1684
50	1.145	251	0.531	25	2.175	1709
100	1.214	298	0.564	30	2.215	1811

Table F12
Armor Weight as Function of Return Period for Station 12 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	0.600	36	0.279	4	1.808	984
10	0.646	44	0.299	4	1.850	1053
15	0.676	51	0.315	5	1.877	1098
20	0.696	56	0.322	6	1.893	1127
25	0.709	60	0.328	6	1.903	1144
30	0.728	64	0.338	6	1.916	1171
35	0.728	64	0.338	6	1.916	1171
40	0.738	67	0.344	7	1.926	1189
45	0.745	69	0.344	7	1.929	1198
50	0.745	69	0.344	7	1.929	1198
100	0.797	84	0.371	8	1.972	1273

Table F13
Armor Weight as Function of Return Period for Station 13 at James Island

Return Period, years	Armor Nominal Diameter D_{n50}, ft	Armor Weight $W_{a,50}$, lb	Filter Layer Nominal Diameter D_{u50}, ft	Filter Layer Weight W_{u50}, lb	Toe Armor Nominal Diameter $D_{toe,50}$, ft	Toe Armor Weight $W_{toe,50}$, ft
5	0.335	7	0.154	1	1.526	591
10	0.417	11	0.194	1	1.634	724
15	0.472	18	0.220	2	1.693	809
20	0.495	20	0.230	2	1.719	847
25	0.528	24	0.246	2	1.752	896
30	0.551	29	0.256	3	1.775	933
35	0.568	31	0.262	3	1.791	956
40	0.581	33	0.269	3	1.804	978
45	0.607	38	0.282	4	1.827	1013
50	0.617	40	0.285	4	1.837	1029
100	0.692	56	0.322	6	1.900	1142

Appendix G Maximum Significant Wave Height for Storm History for Barren Island

Maximum significant wave height by storm, needed to determine return-period wave height values for structure design, was extracted along with corresponding peak wave period, wave direction, and water level. Separate output files were created for tropical storms only, northeasters only, and all storms together. These values of maximum H_s for each storm as well as associated peak period, direction, and water level are tabulated for all stations of Barren Island in this appendix. Tables G1-G6 summarize hurricanes and Tables G7-G12 summarize extratropical storms for Barren Island.

Table G1
Maximum H_s by Storm, Barren Island, Station 1, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	None	1856/08/20	0.47 (1.54)	4.00	254	0.48 (1.57)
2	None	1861/09/26	0.95 (3.12)	4.79	218	0.96 (3.15)
3	None	1861/11/03	0.52 (1.71)	3.22	204	0.38 (1.25)
4	None	1863/09/19	0.69 (2.26)	3.82	210	0.80 (2.62)
5	None	1874/09/28	1.26 (4.13)	6.19	215	1.29 (4.23)
6	None	1876/09/17	1.30 (4.27)	6.28	215	1.35 (4.43)
7	None	1877/10/03	1.18 (3.87)	5.63	210	1.32 (4.33)
8	None	1878/10/22	1.56 (5.12)	6.80	212	1.78 (5.84)
9	None	1879/08/19	0.50 (1.64)	2.68	250	0.87 (2.85)
10	None	1880/09/09	0.81 (2.66)	4.05	207	0.59 (1.94)
11	None	1881/09/10	0.46 (1.51)	3.38	274	0.55 (1.80)
12	None	1888/10/12	0.57 (1.87)	3.48	204	0.62 (2.03)
13	None	1889/09/25	0.77 (2.53)	3.97	202	0.81 (2.66)
14	None	1893/06/17	0.49 (1.61)	3.22	202	0.65 (2.13)
15	None	1893/08/27	1.08 (3.54)	5.75	218	1.08 (3.54)
16	None	1893/10/14	1.05 (3.44)	6.15	219	0.94 (3.08)
17	None	1893/10/21	0.54 (1.77)	3.73	210	0.40 (1.31)
18	None	1894/09/29	0.49 (1.61)	4.01	260	0.61 (2.00)
19	None	1894/10/10	0.67 (2.20)	4.46	210	0.65 (2.13)
20	None	1897/10/25	0.46 (1.51)	3.46	274	0.46 (1.51)
21	None	1899/08/19	0.52 (1.71)	3.99	274	0.93 (3.05)
22	None	1899/10/31	1.22 (4.00)	5.03	214	1.22 (4.00)
23	None	1904/09/14	1.43 (4.69)	6.22	211	1.57 (5.15)
24	None	1908/08/01	0.47 (1.54)	3.38	272	0.39 (1.28)
25	None	1923/10/24	0.58 (1.90)	3.42	213	0.52 (1.71)
26	None	1933/08/24	1.37 (4.49)	5.71	205	1.52 (4.99)
27	None	1933/09/16	0.47 (1.54)	3.78	268	0.64 (2.10)
28	None	1935/09/06	0.69 (2.26)	3.80	202	0.62 (2.03)
29	None	1936/09/19	0.63 (2.07)	4.54	261	0.61 (2.00)
30	None	1944/08/03	1.00 (3.28)	4.89	211	0.90 (2.95)
31	None	1944/09/14	0.52 (1.71)	4.07	263	0.70 (2.30)
32	None	1946/07/07	0.28 (0.92)	2.43	204	0.35 (1.15)
33	Barbara	1953/08/14	0.48 (1.57)	3.87	269	0.67 (2.20)
34	Hazel	1954/10/15	1.38 (4.53)	7.37	217	1.48 (4.86)
35	Connie	1955/08/13	0.93 (3.05)	5.38	221	1.13 (3.71)
36	Diane	1955/08/17	0.87 (2.85)	4.25	218	0.64 (2.10)
37	Ione	1955/09/19	0.49 (1.61)	4.07	254	0.47 (1.54)
38	Brenda	1960/07/30	1.00 (3.28)	4.91	208	0.88 (2.89)
39	Donna	1960/09/12	0.55 (1.80)	4.25	262	0.84 (2.76)
40	Doria	1967/09/11	0.12 (0.39)	1.64	204	0.34 (1.12)
41	Doria	1971/08/28	0.71 (2.33)	3.86	211	0.80 (2.62)
42	Bret	1981/07/01	0.41 (1.35)	3.25	277	0.61 (2.00)
43	Dean	1983/09/30	0.26 (0.85)	2.57	276	0.33 (1.08)
44	Gloria	1985/09/27	0.66 (2.17)	3.03	249	1.04 (3.41)
45	Charley	1986/08/18	0.42 (1.38)	3.89	267	0.45 (1.48)
46	Danielle	1992/09/26	0.24 (0.79)	2.47	276	0.63 (2.07)
47	Bertha	1996/07/13	0.85 (2.79)	4.26	207	0.60 (1.97)
48	Fran	1996/09/06	0.92 (3.02)	4.33	208	1.21 (3.97)
49	Bonnie	1998/08/28	0.46 (1.51)	3.46	280	0.88 (2.89)
50	Earl	1998/09/02	0.55 (1.80)	3.34	212	0.40 (1.31)
51	Floyd	1999/09/16	0.69 (2.26)	3.11	251	1.23 (4.04)
52	Isabel	2003/09/19	1.51 (4.95)	5.95	207	1.85 (6.07)

[1]Storm duration is the time during a storm when H_S > 0.3 m.

Table G2
Maximum H_s by Storm, Barren Island, Station 2, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	None	1856/08/20	0.59 (1.94)	4.00	278	0.48 (1.57)
2	None	1861/09/26	1.07 (3.51)	4.79	226	0.96 (3.15)
3	None	1861/11/03	0.50 (1.64)	3.22	212	0.38 (1.25)
4	None	1863/09/19	0.70 (2.30)	3.82	216	0.80 (2.62)
5	None	1874/09/28	1.43 (4.69)	6.19	222	1.29 (4.23)
6	None	1876/09/17	1.46 (4.79)	6.28	222	1.35 (4.43)
7	None	1877/10/03	1.31 (4.30)	5.63	216	1.32 (4.33)
8	None	1878/10/22	1.72 (5.64)	6.80	218	1.78 (5.84)
9	None	1879/08/19	0.52 (1.71)	2.68	261	0.87 (2.85)
10	None	1880/09/09	0.82 (2.69)	4.05	217	0.59 (1.94)
11	None	1881/09/10	0.44 (1.44)	3.38	285	0.55 (1.80)
12	None	1888/10/12	0.65 (2.13)	3.72	212	0.47 (1.54)
13	None	1889/09/25	0.78 (2.56)	3.97	208	0.81 (2.66)
14	None	1893/06/17	0.49 (1.61)	3.22	207	0.65 (2.13)
15	None	1893/08/27	1.23 (4.04)	5.75	226	1.08 (3.54)
16	None	1893/10/14	1.22 (4.00)	6.15	228	0.94 (3.08)
17	None	1893/10/21	0.65 (2.13)	3.73	217	0.40 (1.31)
18	None	1894/09/29	0.61 (2.00)	4.19	277	0.19 (0.62)
19	None	1894/10/10	0.78 (2.56)	4.46	218	0.65 (2.13)
20	None	1897/10/25	0.56 (1.84)	4.02	279	0.23 (0.75)
21	None	1899/08/19	0.58 (1.90)	3.94	285	0.55 (1.80)
22	None	1899/10/31	1.38 (4.53)	5.86	214	1.21 (3.97)
23	None	1904/09/14	1.59 (5.22)	6.22	217	1.57 (5.15)
24	None	1908/08/01	0.44 (1.44)	3.38	284	0.39 (1.28)
25	None	1923/10/24	0.58 (1.90)	2.77	236	0.40 (1.31)
26	None	1933/08/24	1.49 (4.89)	5.71	210	1.52 (4.99)
27	None	1933/09/16	0.54 (1.77)	3.78	284	0.64 (2.10)
28	None	1935/09/06	0.70 (2.30)	3.80	208	0.62 (2.03)
29	None	1936/09/19	0.77 (2.53)	4.40	276	0.53 (1.74)
30	None	1944/08/03	1.14 (3.74)	4.89	218	0.90 (2.95)
31	None	1944/09/14	0.60 (1.97)	4.07	284	0.70 (2.30)
32	None	1946/07/07	0.27 (0.89)	2.43	212	0.35 (1.15)
33	Barbara	1953/08/14	0.56 (1.84)	3.87	285	0.67 (2.20)
34	Hazel	1954/10/15	1.54 (5.05)	7.37	224	1.48 (4.86)
35	Connie	1955/08/13	1.07 (3.51)	5.38	229	1.13 (3.71)
36	Diane	1955/08/17	0.99 (3.25)	4.90	226	0.67 (2.20)
37	Ione	1955/09/19	0.61 (2.00)	4.07	278	0.47 (1.54)
38	Brenda	1960/07/30	1.14 (3.74)	4.91	215	0.88 (2.89)
39	Donna	1960/09/12	0.64 (2.10)	4.25	283	0.84 (2.76)
40	Doria	1967/09/11	0.11 (0.36)	1.64	212	0.34 (1.12)
41	Doria	1971/08/28	0.72 (2.36)	3.86	217	0.80 (2.62)
42	Bret	1981/07/01	0.45 (1.48)	3.58	287	0.39 (1.28)
43	Dean	1983/09/30	0.25 (0.82)	2.57	288	0.33 (1.08)
44	Gloria	1985/09/27	0.71 (2.33)	4.79	277	1.05 (3.44)
45	Charley	1986/08/18	0.53 (1.74)	3.89	286	0.45 (1.48)
46	Danielle	1992/09/26	0.23 (0.75)	2.47	287	0.63 (2.07)
47	Bertha	1996/07/13	0.92 (3.02)	4.26	217	0.60 (1.97)
48	Fran	1996/09/06	1.01 (3.31)	4.67	215	1.04 (3.41)
49	Bonnie	1998/08/28	0.43 (1.41)	3.46	289	0.88 (2.89)
50	Earl	1998/09/02	0.61 (2.00)	3.64	215	0.54 (1.77)
51	Floyd	1999/09/16	0.72 (2.36)	3.11	262	1.23 (4.04)
52	Isabel	2003/09/19	1.63 (5.35)	5.95	211	1.85 (6.07)

[1] Storm duration is the time during a storm when H_S > 0.3 m.

Table G3
Maximum H_s by Storm, Barren Island, Station 3, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	None	1856/08/20	0.61 (2.00)	4.00	288	0.50 (1.64)
2	None	1861/09/26	1.05 (3.44)	5.42	220	1.03 (3.38)
3	None	1861/11/03	0.46 (1.51)	3.43	296	0.75 (2.46)
4	None	1863/09/19	0.63 (2.07)	3.82	220	0.80 (2.62)
5	None	1874/09/28	1.42 (4.66)	6.19	226	1.31 (4.30)
6	None	1876/09/17	1.47 (4.82)	6.28	226	1.38 (4.53)
7	None	1877/10/03	1.27 (4.17)	5.63	220	1.33 (4.36)
8	None	1878/10/22	1.72 (5.64)	6.80	221	1.81 (5.94)
9	None	1879/08/19	0.54 (1.77)	3.65	293	0.96 (3.15)
10	None	1880/09/09	0.72 (2.36)	4.05	221	0.57 (1.87)
11	None	1881/09/10	0.46 (1.51)	3.38	295	0.57 (1.87)
12	None	1888/10/12	0.58 (1.90)	3.72	217	0.49 (1.61)
13	None	1889/09/25	0.70 (2.30)	3.97	212	0.81 (2.66)
14	None	1893/06/17	0.46 (1.51)	3.51	293	0.55 (1.80)
15	None	1893/08/27	1.24 (4.07)	6.10	232	1.01 (3.31)
16	None	1893/10/14	1.22 (4.00)	6.15	232	0.97 (3.18)
17	None	1893/10/21	0.58 (1.90)	3.73	222	0.40 (1.31)
18	None	1894/09/29	0.65 (2.13)	4.19	287	0.19 (0.62)
19	None	1894/10/10	0.76 (2.49)	4.46	223	0.63 (2.07)
20	None	1897/10/25	0.57 (1.87)	4.02	289	0.23 (0.75)
21	None	1899/08/19	0.60 (1.97)	3.94	294	0.55 (1.80)
22	None	1899/10/31	1.38 (4.53)	6.70	234	1.23 (4.04)
23	None	1904/09/14	1.53 (5.02)	6.22	220	1.58 (5.18)
24	None	1908/08/01	0.47 (1.54)	3.38	294	0.40 (1.31)
25	None	1923/10/24	0.59 (1.94)	3.94	293	0.56 (1.84)
26	None	1933/08/24	1.36 (4.46)	5.71	213	1.52 (4.99)
27	None	1933/09/16	0.57 (1.87)	3.78	293	0.65 (2.13)
28	None	1935/09/06	0.63 (2.07)	3.80	212	0.61 (2.00)
29	None	1936/09/19	0.78 (2.56)	4.40	286	0.52 (1.71)
30	None	1944/08/03	1.05 (3.44)	4.89	222	0.90 (2.95)
31	None	1944/09/14	0.62 (2.03)	4.07	294	0.67 (2.20)
32	None	1946/07/07	0.28 (0.92)	2.67	297	0.48 (1.57)
33	Barbara	1953/08/14	0.58 (1.90)	3.87	294	0.67 (2.20)
34	Hazel	1954/10/15	1.56 (5.12)	7.37	228	1.53 (5.02)
35	Connie	1955/08/13	1.06 (3.48)	5.38	233	1.14 (3.74)
36	Diane	1955/08/17	0.97 (3.18)	4.90	231	0.70 (2.30)
37	Ione	1955/09/19	0.63 (2.07)	4.07	288	0.46 (1.51)
38	Brenda	1960/07/30	1.05 (3.44)	4.91	219	0.89 (2.92)
39	Donna	1960/09/12	0.66 (2.17)	4.25	293	0.82 (2.69)
40	Doria	1967/09/11	0.10 (0.33)	1.64	216	0.35 (1.15)
41	Doria	1971/08/28	0.65 (2.13)	3.86	221	0.81 (2.66)
42	Bret	1981/07/01	0.47 (1.54)	3.58	296	0.39 (1.28)
43	Dean	1983/09/30	0.26 (0.85)	2.57	298	0.35 (1.15)
44	Gloria	1985/09/27	0.74 (2.43)	4.79	286	1.03 (3.38)
45	Charley	1986/08/18	0.54 (1.77)	3.89	295	0.44 (1.44)
46	Danielle	1992/09/26	0.24 (0.79)	2.47	297	0.63 (2.07)
47	Bertha	1996/07/13	0.81 (2.66)	4.26	221	0.57 (1.87)
48	Fran	1996/09/06	0.93 (3.05)	4.67	219	1.04 (3.41)
49	Bonnie	1998/08/28	0.46 (1.51)	3.46	299	0.88 (2.89)
50	Earl	1998/09/02	0.55 (1.80)	3.64	220	0.55 (1.80)
51	Floyd	1999/09/16	0.70 (2.30)	4.43	292	1.26 (4.13)
52	Isabel	2003/09/19	1.47 (4.82)	5.95	214	1.87 (6.14)

[1]Storm duration is the time during a storm when H_S > 0.3 m.

Table G4
Maximum H_s by Storm, Barren Island, Station 4, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	None	1856/08/20	0.51 (1.67)	4.00	278	0.50 (1.64)
2	None	1861/09/26	1.10 (3.61)	5.42	215	1.03 (3.38)
3	None	1861/11/03	0.47 (1.54)	3.22	209	0.36 (1.18)
4	None	1863/09/19	0.64 (2.10)	3.82	214	0.80 (2.62)
5	None	1874/09/28	1.50 (4.92)	6.00	230	1.20 (3.94)
6	None	1876/09/17	1.51 (4.95)	5.92	222	1.38 (4.53)
7	None	1877/10/03	1.27 (4.17)	5.63	215	1.33 (4.36)
8	None	1878/10/22	1.82 (5.97)	6.74	224	1.67 (5.48)
9	None	1879/08/19	0.52 (1.71)	2.68	261	0.88 (2.89)
10	None	1880/09/09	0.73 (2.40)	4.05	215	0.57 (1.87)
11	None	1881/09/10	0.37 (1.21)	3.38	282	0.57 (1.87)
12	None	1888/10/12	0.60 (1.97)	3.72	209	0.49 (1.61)
13	None	1889/09/25	0.71 (2.33)	3.97	206	0.81 (2.66)
14	None	1893/06/17	0.46 (1.51)	3.22	205	0.63 (2.07)
15	None	1893/08/27	1.37 (4.49)	5.75	226	1.01 (3.31)
16	None	1893/10/14	1.39 (4.56)	5.80	226	0.97 (3.18)
17	None	1893/10/21	0.60 (1.97)	3.73	214	0.40 (1.31)
18	None	1894/09/29	0.54 (1.77)	4.19	277	0.19 (0.62)
19	None	1894/10/10	0.79 (2.59)	4.46	216	0.63 (2.07)
20	None	1897/10/25	0.48 (1.57)	4.02	279	0.23 (0.75)
21	None	1899/08/19	0.49 (1.61)	3.94	283	0.55 (1.80)
22	None	1899/10/31	1.56 (5.12)	6.33	228	1.23 (4.04)
23	None	1904/09/14	1.51 (4.95)	5.87	217	1.58 (5.18)
24	None	1908/08/01	0.38 (1.25)	3.51	282	0.54 (1.77)
25	None	1923/10/24	0.57 (1.87)	2.77	237	0.40 (1.31)
26	None	1933/08/24	1.35 (4.43)	5.71	209	1.52 (4.99)
27	None	1933/09/16	0.46 (1.51)	3.78	282	0.65 (2.13)
28	None	1935/09/06	0.64 (2.10)	3.80	206	0.61 (2.00)
29	None	1936/09/19	0.67 (2.20)	4.40	276	0.52 (1.71)
30	None	1944/08/03	1.05 (3.44)	4.89	216	0.90 (2.95)
31	None	1944/09/14	0.51 (1.67)	4.07	283	0.67 (2.20)
32	None	1946/07/07	0.26 (0.85)	2.43	209	0.35 (1.15)
33	Barbara	1953/08/14	0.48 (1.57)	3.87	283	0.67 (2.20)
34	Hazel	1954/10/15	1.74 (5.71)	6.96	224	1.53 (5.02)
35	Connie	1955/08/13	1.10 (3.61)	5.38	228	1.14 (3.74)
36	Diane	1955/08/17	1.02 (3.35)	4.90	224	0.70 (2.30)
37	Ione	1955/09/19	0.52 (1.71)	4.07	278	0.46 (1.51)
38	Brenda	1960/07/30	1.06 (3.48)	4.91	213	0.89 (2.92)
39	Donna	1960/09/12	0.55 (1.80)	3.56	208	0.39 (1.28)
40	Doria	1967/09/11	0.11 (0.36)	1.64	209	0.35 (1.15)
41	Doria	1971/08/28	0.66 (2.17)	3.86	215	0.81 (2.66)
42	Bret	1981/07/01	0.38 (1.25)	3.58	285	0.39 (1.28)
43	Dean	1983/09/30	0.20 (0.66)	2.57	285	0.35 (1.15)
44	Gloria	1985/09/27	0.68 (2.23)	3.03	260	1.03 (3.38)
45	Charley	1986/08/18	0.45 (1.48)	3.89	284	0.44 (1.44)
46	Danielle	1992/09/26	0.19 (0.62)	2.15	214	0.30 (0.98)
47	Bertha	1996/07/13	0.82 (2.69)	4.26	215	0.57 (1.87)
48	Fran	1996/09/06	0.93 (3.05)	4.67	213	1.04 (3.41)
49	Bonnie	1998/08/28	0.37 (1.21)	3.46	287	0.88 (2.89)
50	Earl	1998/09/02	0.57 (1.87)	3.64	212	0.55 (1.80)
51	Floyd	1999/09/16	0.72 (2.36)	3.11	262	1.23 (4.04)
52	Isabel	2003/09/19	1.45 (4.76)	5.95	210	1.87 (6.14)

[1]Storm duration is the time during a storm when H_s > 0.3 m.

Table G5
Maximum H_s by Storm, Barren Island, Station 5, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	None	1856/08/20	0.58 (1.90)	4.00	287	0.50 (1.64)
2	None	1861/09/26	0.98 (3.22)	5.42	220	1.03 (3.38)
3	None	1861/11/03	0.41 (1.35)	3.43	293	0.75 (2.46)
4	None	1863/09/19	0.52 (1.71)	3.82	220	0.80 (2.62)
5	None	1874/09/28	1.29 (4.23)	6.00	237	1.20 (3.94)
6	None	1876/09/17	1.26 (4.13)	5.92	227	1.38 (4.53)
7	None	1877/10/03	1.08 (3.54)	5.63	221	1.33 (4.36)
8	None	1878/10/22	1.57 (5.15)	7.14	229	1.67 (5.48)
9	None	1879/08/19	0.50 (1.64)	2.68	260	0.88 (2.89)
10	None	1880/09/09	0.59 (1.94)	4.05	222	0.57 (1.87)
11	None	1881/09/10	0.41 (1.35)	3.38	292	0.57 (1.87)
12	None	1888/10/12	0.49 (1.61)	3.72	217	0.49 (1.61)
13	None	1889/09/25	0.57 (1.87)	3.97	212	0.81 (2.66)
14	None	1893/06/17	0.44 (1.44)	3.51	293	0.55 (1.80)
15	None	1893/08/27	1.45 (4.76)	6.46	238	0.91 (2.99)
16	None	1893/10/14	1.20 (3.94)	5.80	232	0.97 (3.18)
17	None	1893/10/21	0.49 (1.61)	3.73	222	0.40 (1.31)
18	None	1894/09/29	0.63 (2.07)	4.19	287	0.19 (0.62)
19	None	1894/10/10	0.66 (2.17)	4.46	223	0.63 (2.07)
20	None	1897/10/25	0.54 (1.77)	4.02	288	0.23 (0.75)
21	None	1899/08/19	0.55 (1.80)	3.94	293	0.55 (1.80)
22	None	1899/10/31	1.40 (4.59)	6.33	234	1.23 (4.04)
23	None	1904/09/14	1.25 (4.10)	6.22	222	1.58 (5.18)
24	None	1908/08/01	0.44 (1.44)	3.51	293	0.54 (1.77)
25	None	1923/10/24	0.55 (1.80)	3.94	293	0.56 (1.84)
26	None	1933/08/24	1.12 (3.67)	5.71	215	1.52 (4.99)
27	None	1933/09/16	0.52 (1.71)	3.78	292	0.65 (2.13)
28	None	1935/09/06	0.52 (1.71)	3.80	212	0.61 (2.00)
29	None	1936/09/19	0.74 (2.43)	4.40	285	0.52 (1.71)
30	None	1944/08/03	0.92 (3.02)	5.22	225	1.17 (3.84)
31	None	1944/09/14	0.57 (1.87)	4.07	291	0.67 (2.20)
32	None	1946/07/07	0.24 (0.79)	2.67	295	0.48 (1.57)
33	Barbara	1953/08/14	0.54 (1.77)	3.87	293	0.67 (2.20)
34	Hazel	1954/10/15	1.67 (5.48)	6.96	229	1.53 (5.02)
35	Connie	1955/08/13	0.98 (3.22)	5.38	233	1.14 (3.74)
36	Diane	1955/08/17	0.83 (2.72)	4.90	231	0.70 (2.30)
37	Ione	1955/09/19	0.59 (1.94)	4.07	287	0.46 (1.51)
38	Brenda	1960/07/30	0.84 (2.76)	4.91	220	0.89 (2.92)
39	Donna	1960/09/12	0.61 (2.00)	4.25	290	0.82 (2.69)
40	Doria	1967/09/11	0.08 (0.26)	1.64	217	0.35 (1.15)
41	Doria	1971/08/28	0.53 (1.74)	3.86	221	0.81 (2.66)
42	Bret	1981/07/01	0.44 (1.44)	3.58	296	0.39 (1.28)
43	Dean	1983/09/30	0.23 (0.75)	2.57	296	0.35 (1.15)
44	Gloria	1985/09/27	0.82 (2.69)	4.79	286	1.03 (3.38)
45	Charley	1986/08/18	0.51 (1.67)	3.89	295	0.44 (1.44)
46	Danielle	1992/09/26	0.21 (0.69)	2.47	294	0.63 (2.07)
47	Bertha	1996/07/13	0.66 (2.17)	3.17	238	0.84 (2.76)
48	Fran	1996/09/06	0.75 (2.46)	4.67	220	1.04 (3.41)
49	Bonnie	1998/08/28	0.41 (1.35)	3.46	296	0.88 (2.89)
50	Earl	1998/09/02	0.47 (1.54)	3.64	220	0.55 (1.80)
51	Floyd	1999/09/16	0.68 (2.23)	3.11	261	1.23 (4.04)
52	Isabel	2003/09/19	1.20 (3.94)	5.95	216	1.87 (6.14)

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table G6
Maximum H_s by Storm, Barren Island, Station 6, Tropical Storms

Storm Number	Storm Name	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	None	1856/08/20	0.70 (2.30)	4.00	290	0.48 (1.57)
2	None	1861/09/26	0.85 (2.79)	5.42	224	1.06 (3.48)
3	None	1861/11/03	0.48 (1.57)	3.43	295	0.74 (2.43)
4	None	1863/09/19	0.46 (1.51)	3.30	291	0.65 (2.13)
5	None	1874/09/28	1.14 (3.74)	6.00	240	1.36 (4.46)
6	None	1876/09/17	1.10 (3.61)	6.28	230	1.46 (4.79)
7	None	1877/10/03	0.93 (3.05)	5.63	224	1.37 (4.49)
8	None	1878/10/22	1.40 (4.59)	7.14	231	1.86 (6.10)
9	None	1879/08/19	0.58 (1.90)	3.65	292	0.96 (3.15)
10	None	1880/09/09	0.48 (1.57)	4.05	225	0.53 (1.74)
11	None	1881/09/10	0.48 (1.57)	3.38	294	0.56 (1.84)
12	None	1888/10/12	0.40 (1.31)	3.72	222	0.48 (1.57)
13	None	1889/09/25	0.46 (1.51)	3.97	217	0.80 (2.62)
14	None	1893/06/17	0.52 (1.71)	3.51	294	0.54 (1.77)
15	None	1893/08/27	1.30 (4.27)	6.46	241	1.13 (3.71)
16	None	1893/10/14	1.06 (3.48)	5.80	235	1.09 (3.58)
17	None	1893/10/21	0.43 (1.41)	3.26	295	0.58 (1.90)
18	None	1894/09/29	0.76 (2.49)	4.19	290	0.12 (0.39)
19	None	1894/10/10	0.56 (1.84)	4.46	226	0.58 (1.90)
20	None	1897/10/25	0.66 (2.17)	4.02	291	0.11 (0.36)
21	None	1899/08/19	0.66 (2.17)	3.94	295	0.52 (1.71)
22	None	1899/10/31	1.25 (4.10)	6.33	237	1.42 (4.66)
23	None	1904/09/14	1.08 (3.54)	6.22	224	1.61 (5.28)
24	None	1908/08/01	0.52 (1.71)	3.51	294	0.52 (1.71)
25	None	1923/10/24	0.67 (2.20)	3.94	294	0.56 (1.84)
26	None	1933/08/24	0.96 (3.15)	5.71	218	1.53 (5.02)
27	None	1933/09/16	0.62 (2.03)	3.78	294	0.64 (2.10)
28	None	1935/09/06	0.42 (1.38)	3.80	217	0.60 (1.97)
29	None	1936/09/19	0.91 (2.99)	4.40	288	0.53 (1.74)
30	None	1944/08/03	0.80 (2.62)	5.22	229	1.22 (4.00)
31	None	1944/09/14	0.69 (2.26)	4.07	295	0.63 (2.07)
32	None	1946/07/07	0.28 (0.92)	2.67	296	0.46 (1.51)
33	Barbara	1953/08/14	0.64 (2.10)	3.87	295	0.64 (2.10)
34	Hazel	1954/10/15	1.49 (4.89)	7.37	232	1.73 (5.68)
35	Connie	1955/08/13	0.95 (3.12)	5.06	292	1.32 (4.33)
36	Diane	1955/08/17	0.72 (2.36)	4.90	235	0.77 (2.53)
37	Ione	1955/09/19	0.72 (2.36)	4.07	290	0.42 (1.38)
38	Brenda	1960/07/30	0.72 (2.36)	4.91	224	0.90 (2.95)
39	Donna	1960/09/12	0.74 (2.43)	4.25	294	0.78 (2.56)
40	Doria	1967/09/11	0.07 (0.23)	1.81	198	0.60 (1.97)
41	Doria	1971/08/28	0.58 (1.90)	3.65	292	0.99 (3.25)
42	Bret	1981/07/01	0.53 (1.74)	3.58	297	0.32 (1.05)
43	Dean	1983/09/30	0.26 (0.85)	2.57	297	0.33 (1.08)
44	Gloria	1985/09/27	0.93 (3.05)	4.79	288	1.01 (3.31)
45	Charley	1986/08/18	0.62 (2.03)	3.89	296	0.38 (1.25)
46	Danielle	1992/09/26	0.24 (0.79)	2.47	296	0.63 (2.07)
47	Bertha	1996/07/13	0.61 (2.00)	3.17	242	0.88 (2.89)
48	Fran	1996/09/06	0.63 (2.07)	4.67	224	1.05 (3.44)
49	Bonnie	1998/08/28	0.48 (1.57)	3.46	298	0.86 (2.82)
50	Earl	1998/09/02	0.38 (1.25)	3.64	225	0.55 (1.80)
51	Floyd	1999/09/16	0.78 (2.56)	4.43	292	1.22 (4.00)
52	Isabel	2003/09/19	1.03 (3.38)	5.95	219	1.89 (6.20)

[1]Storm duration is the time during a storm when $H_S > 0.3$ m.

Table G7
Maximum H_s by Storm, Barren Island, Station 1, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	1954/01/23	0.59 (1.94)	4.46	254	0.32 (1.05)
2	1956/10/17	0.47 (1.54)	4.06	257	0.50 (1.64)
3	1956/10/28	0.64 (2.10)	4.62	260	0.65 (2.13)
4	1957/10/06	0.66 (2.17)	4.70	260	0.59 (1.94)
5	1958/02/17	0.72 (2.36)	4.26	211	0.38 (1.25)
6	1958/10/21	0.54 (1.77)	4.22	260	0.56 (1.84)
7	1962/03/08	0.67 (2.20)	4.74	260	0.67 (2.20)
8	1962/11/27	0.53 (1.74)	4.02	266	1.04 (3.41)
9	1966/01/31	0.68 (2.23)	4.48	258	0.52 (1.71)
10	1969/01/22	0.47 (1.54)	3.46	273	0.11 (0.36)
11	1972/05/26	0.56 (1.84)	3.78	209	0.43 (1.41)
12	1972/10/08	0.61 (2.00)	4.22	257	0.81 (2.66)
13	1974/12/04	1.10 (3.61)	5.10	216	1.07 (3.51)
14	1975/07/01	0.61 (2.00)	4.56	254	0.42 (1.38)
15	1977/10/30	0.47 (1.54)	3.51	274	0.50 (1.64)
16	1978/04/28	0.63 (2.07)	4.41	259	0.54 (1.77)
17	1980/12/30	0.55 (1.80)	4.06	258	0.48 (1.57)
18	1981/08/21	0.56 (1.84)	3.43	205	0.63 (2.07)
19	1983/02/12	0.61 (2.00)	4.25	258	0.59 (1.94)
20	1984/03/30	1.02 (3.35)	4.95	217	0.92 (3.02)
21	1984/09/30	0.49 (1.61)	3.86	268	0.73 (2.39)
22	1984/10/14	0.58 (1.90)	4.24	259	0.77 (2.53)
23	1984/11/21	0.49 (1.61)	4.07	254	0.28 (0.92)
24	1985/10/29	0.91 (2.99)	4.27	206	0.99 (3.25)
25	1986/12/01	0.61 (2.00)	3.56	199	1.03 (3.38)
26	1987/02/18	0.47 (1.54)	3.51	274	0.26 (0.85)
27	1988/04/14	0.49 (1.61)	4.01	260	0.62 (2.03)
28	1989/03/10	0.48 (1.57)	3.87	269	0.56 (1.84)
29	1991/01/09	0.46 (1.51)	3.38	274	0.66 (2.17)
30	1991/04/21	0.47 (1.54)	3.38	272	0.52 (1.71)
31	1991/10/31	0.48 (1.57)	3.87	269	0.64 (2.10)
32	1991/11/10	0.48 (1.57)	3.96	270	0.55 (1.80)
33	1993/03/15	0.68 (2.23)	3.05	250	0.82 (2.69)
34	1994/10/16	0.47 (1.54)	3.52	274	0.42 (1.38)
35	1996/10/09	0.48 (1.57)	3.79	265	0.64 (2.10)
36	1997/06/04	0.45 (1.48)	3.44	276	0.58 (1.90)
37	1997/10/16	0.47 (1.54)	3.40	271	0.46 (1.51)
38	1998/05/13	0.47 (1.54)	3.52	276	0.71 (2.33)
39	1999/05/03	0.47 (1.54)	3.51	274	0.46 (1.51)
40	1999/08/31	0.66 (2.17)	3.72	202	0.60 (1.97)
41	2000/05/30	0.48 (1.57)	3.53	275	0.55 (1.80)
42	2003/04/11	0.49 (1.61)	3.86	268	0.76 (2.49)
43	2003/09/10	0.43 (1.41)	3.32	275	0.44 (1.44)

[1]Storm duration is the time during a storm when $H_s > 0.3$ m.

Table G8
Maximum H_s by Storm, Barren Island, Station 2, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	1954/01/23	0.74 (2.43)	4.46	278	0.32 (1.05)
2	1956/10/17	0.58 (1.90)	4.06	281	0.50 (1.64)
3	1956/10/28	0.75 (2.46)	4.62	281	0.65 (2.13)
4	1957/10/06	0.78 (2.56)	4.70	281	0.59 (1.94)
5	1958/02/17	0.88 (2.89)	4.26	221	0.38 (1.25)
6	1958/10/21	0.63 (2.07)	4.22	281	0.56 (1.84)
7	1962/03/08	0.80 (2.62)	4.82	279	0.52 (1.71)
8	1962/11/27	0.65 (2.13)	4.12	277	0.54 (1.77)
9	1966/01/31	0.81 (2.66)	4.17	233	0.27 (0.89)
10	1969/01/22	0.47 (1.54)	3.62	289	0.59 (1.94)
11	1972/05/26	0.68 (2.23)	3.78	216	0.43 (1.41)
12	1972/10/08	0.74 (2.43)	4.29	275	0.43 (1.41)
13	1974/12/04	1.25 (4.10)	5.10	223	1.07 (3.51)
14	1975/07/01	0.77 (2.53)	4.56	278	0.42 (1.38)
15	1977/10/30	0.53 (1.74)	3.90	285	0.37 (1.21)
16	1978/04/28	0.73 (2.39)	4.41	280	0.54 (1.77)
17	1980/12/30	0.64 (2.10)	4.06	279	0.48 (1.57)
18	1981/08/21	0.62 (2.03)	4.22	279	0.57 (1.87)
19	1983/02/12	0.71 (2.33)	4.25	279	0.59 (1.94)
20	1984/03/30	1.17 (3.84)	4.95	224	0.92 (3.02)
21	1984/09/30	0.57 (1.87)	3.86	284	0.73 (2.39)
22	1984/10/14	0.68 (2.23)	4.24	280	0.77 (2.53)
23	1984/11/21	0.62 (2.03)	4.12	276	0.23 (0.75)
24	1985/10/29	0.90 (2.95)	4.27	214	0.99 (3.25)
25	1986/12/01	0.60 (1.97)	3.56	203	1.03 (3.38)
26	1987/02/18	0.47 (1.54)	3.72	285	0.33 (1.08)
27	1988/04/14	0.56 (1.84)	4.01	281	0.62 (2.03)
28	1989/03/10	0.56 (1.84)	3.87	285	0.56 (1.84)
29	1991/01/09	0.48 (1.57)	3.72	286	0.18 (0.59)
30	1991/04/21	0.44 (1.44)	3.38	284	0.52 (1.71)
31	1991/10/31	0.56 (1.84)	3.87	285	0.64 (2.10)
32	1991/11/10	0.55 (1.80)	3.96	287	0.55 (1.80)
33	1993/03/15	0.70 (2.30)	3.05	264	0.82 (2.69)
34	1994/10/16	0.50 (1.64)	3.78	286	0.55 (1.80)
35	1996/10/09	0.56 (1.84)	3.79	281	0.64 (2.10)
36	1997/06/04	0.43 (1.41)	3.44	287	0.58 (1.90)
37	1997/10/16	0.49 (1.61)	3.66	285	0.60 (1.97)
38	1998/05/13	0.45 (1.48)	3.52	288	0.71 (2.33)
39	1999/05/03	0.47 (1.54)	3.72	285	0.51 (1.67)
40	1999/08/31	0.67 (2.20)	3.72	208	0.60 (1.97)
41	2000/05/30	0.53 (1.74)	3.90	285	0.58 (1.90)
42	2003/04/11	0.57 (1.87)	3.86	284	0.76 (2.49)
43	2003/09/10	0.44 (1.44)	3.59	286	0.40 (1.31)

[1]Storm duration is the time during a storm when H_s > 0.3 m.

Table G9
Maximum H_s by Storm, Barren Island, Station 3, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	1954/01/23	0.75 (2.46)	4.46	288	0.29 (0.95)
2	1956/10/17	0.60 (1.97)	4.06	291	0.50 (1.64)
3	1956/10/28	0.75 (2.46)	4.62	292	0.65 (2.13)
4	1957/10/06	0.78 (2.56)	4.70	292	0.59 (1.94)
5	1958/02/17	0.80 (2.62)	4.26	224	0.37 (1.21)
6	1958/10/21	0.64 (2.10)	4.22	292	0.55 (1.80)
7	1962/03/08	0.80 (2.62)	4.82	289	0.51 (1.67)
8	1962/11/27	0.67 (2.20)	4.12	287	0.52 (1.71)
9	1966/01/31	0.80 (2.62)	4.48	289	0.50 (1.64)
10	1969/01/22	0.49 (1.61)	3.62	298	0.56 (1.84)
11	1972/05/26	0.61 (2.00)	3.78	221	0.42 (1.38)
12	1972/10/08	0.76 (2.49)	4.29	284	0.42 (1.38)
13	1974/12/04	1.14 (3.74)	5.10	227	1.07 (3.51)
14	1975/07/01	0.78 (2.56)	4.56	288	0.39 (1.28)
15	1977/10/30	0.54 (1.77)	3.90	294	0.37 (1.21)
16	1978/04/28	0.74 (2.43)	4.41	290	0.55 (1.80)
17	1980/12/30	0.66 (2.17)	4.06	289	0.47 (1.54)
18	1981/08/21	0.63 (2.07)	4.22	290	0.54 (1.77)
19	1983/02/12	0.72 (2.36)	4.25	289	0.58 (1.90)
20	1984/03/30	1.07 (3.51)	4.95	228	0.92 (3.02)
21	1984/09/30	0.59 (1.94)	3.86	293	0.74 (2.43)
22	1984/10/14	0.69 (2.26)	4.24	290	0.75 (2.46)
23	1984/11/21	0.66 (2.17)	4.12	286	0.21 (0.69)
24	1985/10/29	0.81 (2.66)	4.27	218	1.00 (3.28)
25	1986/12/01	0.55 (1.80)	3.56	206	1.05 (3.44)
26	1987/02/18	0.49 (1.61)	3.72	294	0.33 (1.08)
27	1988/04/14	0.58 (1.90)	4.01	292	0.63 (2.07)
28	1989/03/10	0.58 (1.90)	3.87	294	0.57 (1.87)
29	1991/01/09	0.50 (1.64)	3.72	295	0.16 (0.52)
30	1991/04/21	0.47 (1.54)	3.38	294	0.51 (1.67)
31	1991/10/31	0.58 (1.90)	3.87	294	0.62 (2.03)
32	1991/11/10	0.57 (1.87)	3.96	296	0.52 (1.71)
33	1993/03/15	0.71 (2.33)	3.44	263	0.86 (2.82)
34	1994/10/16	0.52 (1.71)	3.78	295	0.52 (1.71)
35	1996/10/09	0.58 (1.90)	3.79	290	0.65 (2.13)
36	1997/06/04	0.45 (1.48)	3.44	297	0.59 (1.94)
37	1997/10/16	0.52 (1.71)	3.66	294	0.57 (1.87)
38	1998/05/13	0.47 (1.54)	3.52	298	0.70 (2.30)
39	1999/05/03	0.49 (1.61)	3.72	294	0.51 (1.67)
40	1999/08/31	0.61 (2.00)	4.13	289	0.43 (1.41)
41	2000/05/30	0.54 (1.77)	3.90	294	0.55 (1.80)
42	2003/04/11	0.59 (1.94)	3.86	293	0.74 (2.43)
43	2003/09/10	0.47 (1.54)	3.59	295	0.38 (1.25)

[1]Storm duration is the time during a storm when H_s > 0.3 m.

Table G10
Maximum H_s by Storm, Barren Island, Station 4, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	1954/01/23	0.70 (2.30)	4.82	279	0.13 (0.43)
2	1956/10/17	0.50 (1.64)	4.06	281	0.50 (1.64)
3	1956/10/28	0.65 (2.13)	4.62	280	0.65 (2.13)
4	1957/10/06	0.67 (2.20)	4.70	280	0.59 (1.94)
5	1958/02/17	0.81 (2.66)	4.26	217	0.37 (1.21)
6	1958/10/21	0.54 (1.77)	4.22	280	0.55 (1.80)
7	1962/03/08	0.70 (2.30)	4.82	279	0.51 (1.67)
8	1962/11/27	0.56 (1.84)	4.12	277	0.52 (1.71)
9	1966/01/31	0.77 (2.53)	4.17	229	0.27 (0.89)
10	1969/01/22	0.40 (1.31)	3.62	287	0.56 (1.84)
11	1972/05/26	0.62 (2.03)	3.78	213	0.42 (1.38)
12	1972/10/08	0.64 (2.10)	4.29	275	0.42 (1.38)
13	1974/12/04	1.15 (3.77)	5.10	221	1.07 (3.51)
14	1975/07/01	0.67 (2.20)	4.56	278	0.39 (1.28)
15	1977/10/30	0.45 (1.48)	3.90	283	0.37 (1.21)
16	1978/04/28	0.63 (2.07)	4.41	279	0.55 (1.80)
17	1980/12/30	0.55 (1.80)	4.06	278	0.47 (1.54)
18	1981/08/21	0.53 (1.74)	4.22	279	0.54 (1.77)
19	1983/02/12	0.60 (1.97)	4.25	278	0.58 (1.90)
20	1984/03/30	1.08 (3.54)	4.95	222	0.92 (3.02)
21	1984/09/30	0.52 (1.71)	3.48	219	0.34 (1.12)
22	1984/10/14	0.59 (1.94)	4.43	279	0.52 (1.71)
23	1984/11/21	0.55 (1.80)	4.12	276	0.21 (0.69)
24	1985/10/29	0.81 (2.66)	4.27	213	1.00 (3.28)
25	1986/12/01	0.56 (1.84)	3.56	202	1.05 (3.44)
26	1987/02/18	0.41 (1.35)	3.72	283	0.33 (1.08)
27	1988/04/14	0.48 (1.57)	4.01	280	0.63 (2.07)
28	1989/03/10	0.48 (1.57)	3.87	283	0.57 (1.87)
29	1991/01/09	0.41 (1.35)	3.72	283	0.16 (0.52)
30	1991/04/21	0.37 (1.21)	3.38	281	0.51 (1.67)
31	1991/10/31	0.48 (1.57)	3.87	283	0.62 (2.03)
32	1991/11/10	0.47 (1.54)	3.96	285	0.52 (1.71)
33	1993/03/15	0.71 (2.33)	3.44	262	0.86 (2.82)
34	1994/10/16	0.42 (1.38)	3.78	283	0.52 (1.71)
35	1996/10/09	0.48 (1.57)	3.79	279	0.65 (2.13)
36	1997/06/04	0.38 (1.25)	2.97	209	0.78 (2.56)
37	1997/10/16	0.42 (1.38)	3.66	283	0.57 (1.87)
38	1998/05/13	0.38 (1.25)	3.52	285	0.70 (2.30)
39	1999/05/03	0.41 (1.35)	3.72	283	0.51 (1.67)
40	1999/08/31	0.61 (2.00)	3.72	206	0.59 (1.94)
41	2000/05/30	0.45 (1.48)	3.90	283	0.55 (1.80)
42	2003/04/11	0.48 (1.57)	3.86	282	0.74 (2.43)
43	2003/09/10	0.38 (1.25)	3.59	283	0.38 (1.25)

[1]Storm duration is the time during a storm when H_s > 0.3 m.

Table G11
Maximum H_s by Storm, Barren Island, Station 5, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	1954/01/23	0.89 (2.92)	5.14	286	0.31 (1.02)
2	1956/10/17	0.57 (1.87)	4.06	290	0.50 (1.64)
3	1956/10/28	0.71 (2.33)	4.62	289	0.65 (2.13)
4	1957/10/06	0.74 (2.43)	4.70	289	0.59 (1.94)
5	1958/02/17	0.74 (2.43)	4.35	283	0.36 (1.18)
6	1958/10/21	0.60 (1.97)	4.22	289	0.55 (1.80)
7	1962/03/08	0.77 (2.53)	4.82	288	0.51 (1.67)
8	1962/11/27	0.63 (2.07)	4.12	286	0.52 (1.71)
9	1966/01/31	0.76 (2.49)	3.29	261	-0.17 (-0.56)
10	1969/01/22	0.45 (1.48)	3.62	297	0.56 (1.84)
11	1972/05/26	0.53 (1.74)	3.94	295	0.65 (2.13)
12	1972/10/08	0.74 (2.43)	4.53	282	0.49 (1.61)
13	1974/12/04	0.91 (2.99)	5.10	228	1.07 (3.51)
14	1975/07/01	0.75 (2.46)	4.75	286	0.23 (0.75)
15	1977/10/30	0.51 (1.67)	3.90	294	0.37 (1.21)
16	1978/04/28	0.70 (2.30)	4.41	287	0.55 (1.80)
17	1980/12/30	0.61 (2.00)	4.06	286	0.43 (1.41)
18	1981/08/21	0.60 (1.97)	4.22	289	0.54 (1.77)
19	1983/02/12	0.67 (2.20)	4.25	286	0.40 (1.31)
20	1984/03/30	1.00 (3.28)	5.10	283	0.60 (1.97)
21	1984/09/30	0.55 (1.80)	3.86	292	0.74 (2.43)
22	1984/10/14	0.66 (2.17)	4.43	288	0.52 (1.71)
23	1984/11/21	0.63 (2.07)	4.12	286	0.21 (0.69)
24	1985/10/29	0.66 (2.17)	4.27	220	1.00 (3.28)
25	1986/12/01	0.50 (1.64)	3.84	295	0.39 (1.28)
26	1987/02/18	0.47 (1.54)	3.72	294	0.33 (1.08)
27	1988/04/14	0.54 (1.77)	4.01	289	0.63 (2.07)
28	1989/03/10	0.54 (1.77)	3.87	293	0.57 (1.87)
29	1991/01/09	0.47 (1.54)	3.72	295	0.16 (0.52)
30	1991/04/21	0.41 (1.35)	3.38	292	0.51 (1.67)
31	1991/10/31	0.54 (1.77)	3.87	293	0.62 (2.03)
32	1991/11/10	0.53 (1.74)	3.96	295	0.52 (1.71)
33	1993/03/15	0.76 (2.49)	3.44	261	0.86 (2.82)
34	1994/10/16	0.49 (1.61)	3.78	295	0.52 (1.71)
35	1996/10/09	0.55 (1.80)	3.94	292	0.21 (0.69)
36	1997/06/04	0.40 (1.31)	3.44	294	0.59 (1.94)
37	1997/10/16	0.48 (1.57)	3.66	293	0.57 (1.87)
38	1998/05/13	0.42 (1.38)	3.52	295	0.70 (2.30)
39	1999/05/03	0.47 (1.54)	3.72	294	0.51 (1.67)
40	1999/08/31	0.57 (1.87)	4.13	288	0.43 (1.41)
41	2000/05/30	0.51 (1.67)	3.90	294	0.55 (1.80)
42	2003/04/11	0.55 (1.80)	3.86	292	0.74 (2.43)
43	2003/09/10	0.44 (1.44)	3.59	295	0.38 (1.25)

[1]Storm duration is the time during a storm when $H_s > 0.3$ m.

Table G12
Maximum H_s by Storm, Barren Island, Station 6, Extratropical Storms

Storm Number	Date	H_s, m (ft)	T_p, sec	θ_p, deg az.	Water Level, m (ft) mllw
1	1954/01/23	0.98 (3.22)	5.14	287	0.04 (0.13)
2	1956/10/17	0.69 (2.26)	4.06	293	0.40 (1.31)
3	1956/10/28	0.87 (2.85)	4.62	293	0.53 (1.74)
4	1957/10/06	0.90 (2.95)	4.70	293	0.43 (1.41)
5	1958/02/17	0.91 (2.99)	4.35	286	0.35 (1.15)
6	1958/10/21	0.73 (2.39)	4.22	292	0.29 (0.95)
7	1962/03/08	0.96 (3.15)	4.82	291	0.30 (0.98)
8	1962/11/27	0.77 (2.53)	4.12	289	0.34 (1.12)
9	1966/01/31	0.91 (2.99)	4.48	290	0.41 (1.35)
10	1969/01/22	0.53 (1.74)	3.62	299	0.50 (1.64)
11	1972/05/26	0.63 (2.07)	3.94	297	0.58 (1.90)
12	1972/10/08	0.88 (2.89)	4.29	286	0.43 (1.41)
13	1974/12/04	0.90 (2.95)	4.35	287	0.52 (1.71)
14	1975/07/01	0.92 (3.02)	4.56	290	0.00 (0.00)
15	1977/10/30	0.62 (2.03)	3.90	295	0.23 (0.75)
16	1978/04/28	0.85 (2.79)	4.31	289	0.30 (0.98)
17	1980/12/30	0.74 (2.43)	4.06	289	0.37 (1.21)
18	1981/08/21	0.73 (2.39)	4.22	292	0.33 (1.08)
19	1983/02/12	0.82 (2.69)	4.25	289	0.38 (1.25)
20	1984/03/30	1.12 (3.67)	5.10	285	0.59 (1.94)
21	1984/09/30	0.66 (2.17)	3.86	293	0.32 (1.05)
22	1984/10/14	0.81 (2.66)	4.43	291	0.37 (1.21)
23	1984/11/21	0.76 (2.49)	4.12	289	0.00 (0.00)
24	1985/10/29	0.57 (1.87)	3.63	293	0.31 (1.02)
25	1986/12/01	0.60 (1.97)	3.84	296	0.30 (0.98)
26	1987/02/18	0.56 (1.84)	3.72	295	0.21 (0.69)
27	1988/04/14	0.65 (2.13)	4.01	293	0.57 (1.87)
28	1989/03/10	0.64 (2.10)	3.87	295	0.51 (1.67)
29	1991/01/09	0.56 (1.84)	3.72	296	0.00 (0.00)
30	1991/04/21	0.48 (1.57)	3.38	293	0.43 (1.41)
31	1991/10/31	0.64 (2.10)	3.87	295	0.55 (1.80)
32	1991/11/10	0.63 (2.07)	3.96	297	0.43 (1.41)
33	1993/03/15	0.83 (2.72)	4.49	283	0.64 (2.10)
34	1994/10/16	0.58 (1.90)	3.78	296	0.38 (1.25)
35	1996/10/09	0.66 (2.17)	3.94	294	0.00 (0.00)
36	1997/06/04	0.47 (1.54)	3.44	296	0.55 (1.80)
37	1997/10/16	0.56 (1.84)	3.66	295	0.55 (1.80)
38	1998/05/13	0.49 (1.61)	3.52	297	0.65 (2.13)
39	1999/05/03	0.56 (1.84)	3.72	295	0.43 (1.41)
40	1999/08/31	0.70 (2.30)	4.13	291	0.38 (1.25)
41	2000/05/30	0.62 (2.03)	3.90	295	0.42 (1.38)
42	2003/04/11	0.65 (2.13)	3.86	294	0.71 (2.33)
43	2003/09/10	0.52 (1.71)	3.59	296	0.27 (0.89)

[1]Storm duration is the time during a storm when $H_s > 0.3$ m.

Appendix H
Extremal Wave and Water Level Analysis Results for Barren Island

Extremal analysis of significant wave heights was applied to all storms together and to hurricanes only, and results are summarized in this appendix. Analysis of all storms included 179 storms over the 148-year time period. Analysis of hurricanes only included 52 storms over the 148-year period. The best-fitting extremal distribution was selected, based on the criteria of Goda and Kobune (1990) and a good visual fit to the return periods of concern for this project. Using the best-fit distribution, significant wave heights were determined for return periods of 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, and 100 years. For hurricane-influenced stations where the best-fit distribution for all storms underestimated H_s at the longest return periods, return period H_s was taken from the best fit for hurricanes only for return periods dominated by hurricanes.

To estimate an appropriate peak wave period and water level to accompany each return-period significant wave height, the computer program return_period_Tp.f is run. Inputs include return-period significant wave heights and 148-year time history of waves and water levels at each station. The time history is screened to find all significant heights within a bin centered on the desired return-period wave height. Bin widths considered are 0.2, 0.4, 0.6, 0.8, and 1.0 m (0.7, 1.3, 2.0, 2.6, and 3.3 ft). For each return period, a representative or *average* period and water level were chosen with consideration of bins that captured enough cases to form a meaningful average but not so many cases as to dilute the target severe events.

Tables H1-H6 summarize extremal wave height analysis results for sta 1-6 of Barren Island. The extremal values are plotted as a function of return period in Figures H1-H12. Figures H2, H4, H6, H8, H10, and H12 show the bin-averaged water levels corresponding to extremal wave heights superimposed on several freeboard levels, all relative to mllw.

Tables H7 and H8 give results of an extremal analysis of storm water levels for Barren Island. The maximum water levels for each storm were fit to a Fisher-Tippett type I distribution. The extremal water levels, referenced to msl, associated with northeasters from Table 16 in Chapter 3 are listed in Table H7. The extremal water levels, referenced to msl, associated with tropical storms from Table 13 in Chapter 3 are listed in Table H8. The relationship used here for Barren Island tidal datums is msl = 0.240 m mllw. As described for Poplar and James Islands, this extreme water level analysis only included storms.

Table H1
Barren Island Station 1 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.81 (2.66)	4.29	0.85 (2.79)
10	0.97 (3.18)	4.89	0.96 (3.15)
15	1.07 (3.51)	5.26	1.01 (3.31)
20	1.14 (3.74)	5.47	1.18 (3.87)
25	1.20 (3.94)	5.66	1.35 (4.43)
30	1.26 (4.13)	5.70	1.35 (4.43)
35	1.31 (4.30)	5.90	1.45 (4.76)
40	1.35 (4.43)	6.08	1.49 (4.89)
45	1.38 (4.53)	6.30	1.53 (5.02)
50	1.41 (4.63)	6.38	1.61 (5.28)
100	1.61 (5.28)	6.38	1.82 (5.97)

Table H2
Barren Island Station 2 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height *Hs*, m (ft)	Peak Wave Period *Tp*, sec	Water Level mllw, m (ft)
5	0.91 (2.99)	4.37	0.80 (2.62)
10	1.10 (3.61)	5.11	0.98 (3.22)
15	1.21 (3.97)	5.23	1.06 (3.48)
20	1.29 (4.23)	5.52	1.16 (3.81)
25	1.35 (4.43)	5.55	1.27 (4.17)
30	1.41 (4.63)	5.90	1.34 (4.40)
35	1.46 (4.79)	6.16	1.45 (4.76)
40	1.51 (4.95)	6.20	1.49 (4.89)
45	1.55 (5.09)	6.34	1.56 (5.12)
50	1.59 (5.22)	6.38	1.61 (5.28)
100	1.79 (5.87)	6.80	1.78 (5.84)

Table H3
Barren Island Station 3 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.84 (2.76)	4.47	0.65 (2.13)
10	1.02 (3.35)	4.97	0.99 (3.25)
15	1.14 (3.74)	5.26	1.10 (3.61)
20	1.22 (4.00)	5.55	1.20 (3.94)
25	1.29 (4.23)	5.88	1.33 (4.36)
30	1.36 (4.46)	5.97	1.40 (4.59)
35	1.41 (4.63)	6.10	1.44 (4.72)
40	1.45 (4.76)	6.15	1.46 (4.79)
45	1.49 (4.89)	6.39	1.51 (4.95)
50	1.52 (4.99)	6.42	1.54 (5.05)
100	1.73 (5.68)	6.97	1.74 (5.71)

Table H4
Barren Island Station 4 Extremal Wave Analysis Results

Return Period, years	Significant Wave Height H_s, m (ft)	Peak Wave Period T_p, sec	Water Level mllw, m (ft)
5	0.87 (2.85)	4.34	0.76 (2.49)
10	1.07 (3.51)	4.98	1.02 (3.35)
15	1.18 (3.87)	5.30	1.15 (3.77)
20	1.27 (4.17)	5.72	1.32 (4.33)
25	1.33 (4.36)	5.79	1.25 (4.10)
30	1.39 (4.56)	5.85	1.36 (4.46)
35	1.43 (4.69)	5.89	1.35 (4.43)
40	1.47 (4.82)	5.91	1.32 (4.33)
45	1.50 (4.92)	5.96	1.43 (4.69)
50	1.53 (5.02)	5.96	1.43 (4.69)
100	1.74 (5.71)	6.57	1.62 (5.31)

Table H5 **Barren Island Station 5 Extremal Wave Analysis Results**			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.83 (2.72)	4.59	0.54 (1.77)
10	0.99 (3.25)	5.10	0.95 (3.12)
15	1.08 (3.54)	5.42	1.15 (3.77)
20	1.15 (3.77)	5.76	1.34 (4.40)
25	1.20 (3.94)	5.86	1.33 (4.36)
30	1.25 (4.10)	5.91	1.35 (4.43)
35	1.30 (4.27)	6.09	1.37 (4.49)
40	1.35 (4.43)	6.30	1.45 (4.76)
45	1.38 (4.53)	6.42	1.32 (4.33)
50	1.41 (4.63)	6.53	1.36 (4.46)
100	1.61 (5.28)	7.05	1.60 (5.25)

Table H6 **Barren Island Station 6 Extremal Wave Analysis Results**			
Return Period, years	**Significant Wave Height H_s, m (ft)**	**Peak Wave Period T_p, sec**	**Water Level mllw, m (ft)**
5	0.90 (2.95)	4.48	0.30 (0.98)
10	1.02 (3.35)	5.25	0.78 (2.56)
15	1.08 (3.54)	5.49	1.04 (3.41)
20	1.13 (3.71)	5.53	1.08 (3.54)
25	1.17 (3.84)	5.77	1.16 (3.81)
30	1.20 (3.94)	5.90	1.17 (3.84)
35	1.22 (4.00)	6.42	1.45 (4.76)
40	1.25 (4.10)	6.53	1.47 (4.82)
45	1.26 (4.13)	6.53	1.47 (4.82)
50	1.28 (4.20)	6.53	1.46 (4.79)
100	1.44 (4.72)	7.26	1.80 (5.91)

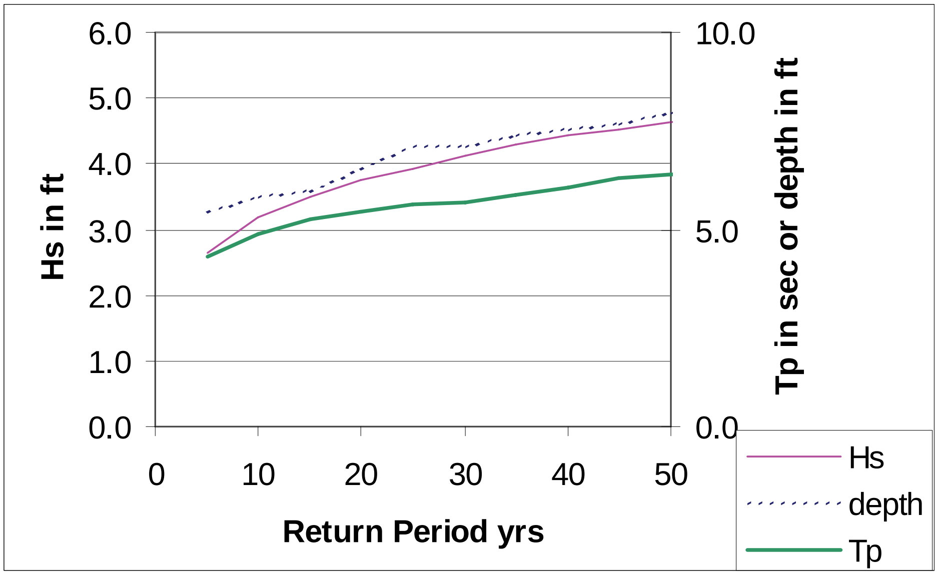

Figure H1. Barren Island sta 1 significant wave height, peak period, and depth (mllw) as function of return period (1 ft = 0.3048 m)

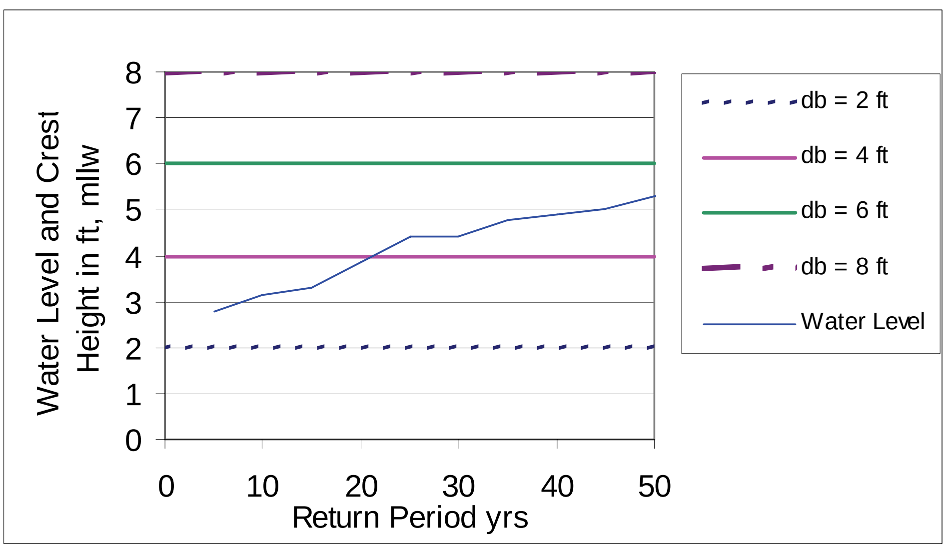

Figure H2. Barren Island sta 1 water level and crest height as function of return period (1 ft = 0.3048 m)

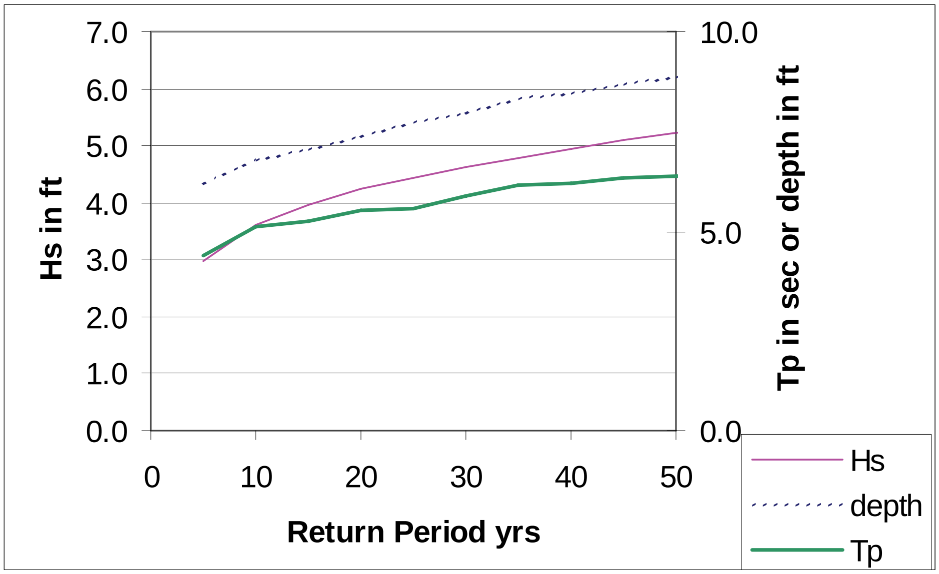

Figure H3. Barren Island sta 2 significant wave height, peak period, and depth (mllw) as function of return period (1 ft = 0.3048 m)

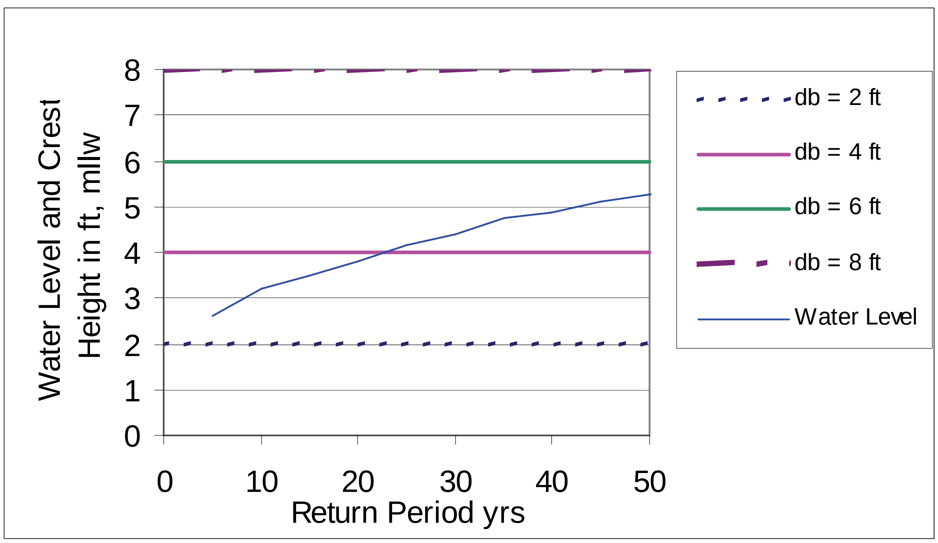

Figure H4. Barren Island sta 2 water level and crest height as function of return period (1 ft = 0.3048 m)

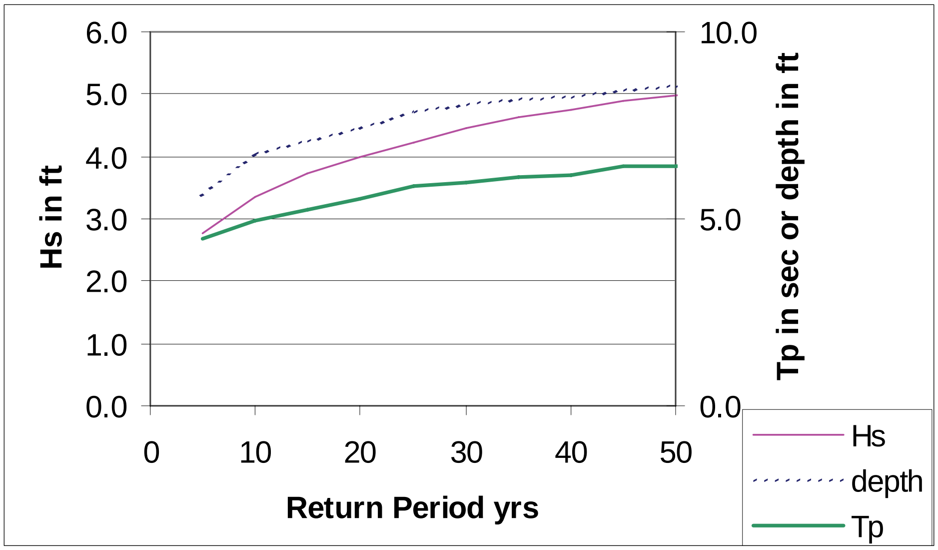

Figure H5. Barren Island sta 3 significant wave height, peak period, and depth (mllw) as function of return period (1 ft = 0.3048 m)

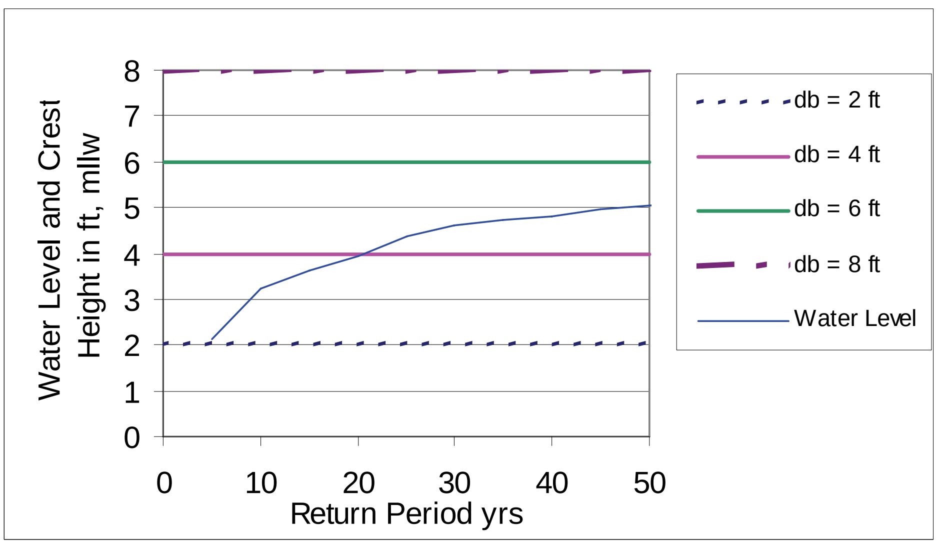

Figure H6. Barren Island sta 3 water level and crest height as function of return period (1 ft = 0.3048 m)

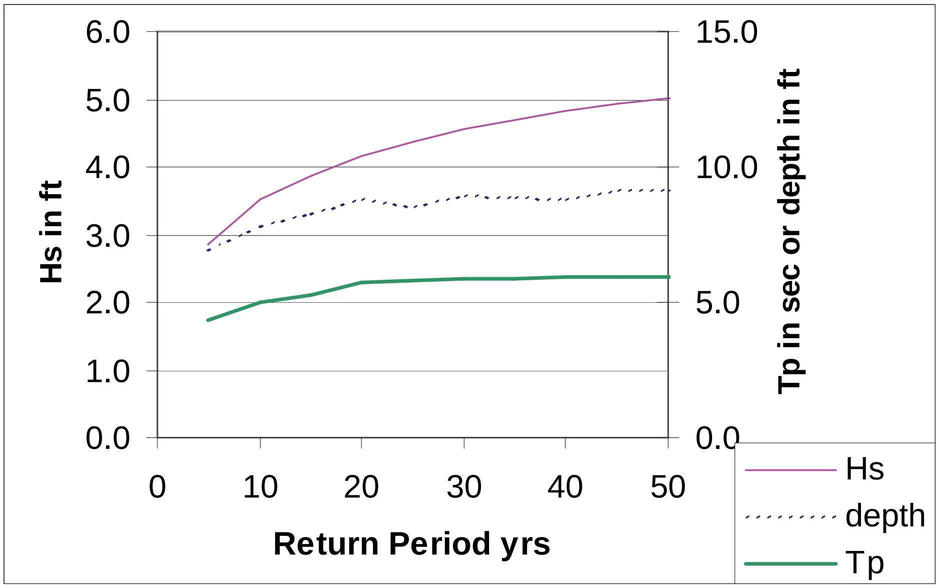

Figure H7. Barren Island sta 4 significant wave height, peak period, and depth (mllw) as function of return period (1 ft = 0.3048 m)

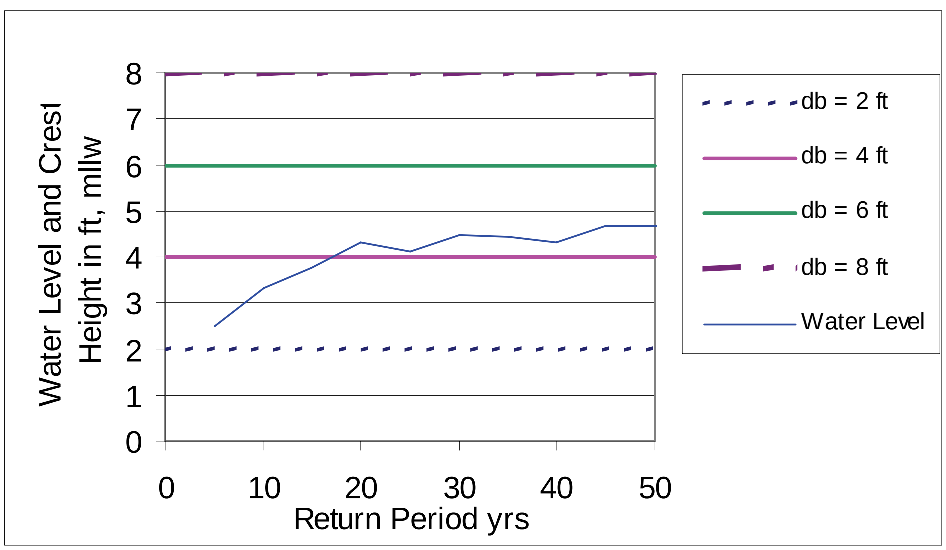

Figure H8. Barren Island sta 4 water level and crest height as function of return period (1 ft = 0.3048 m)

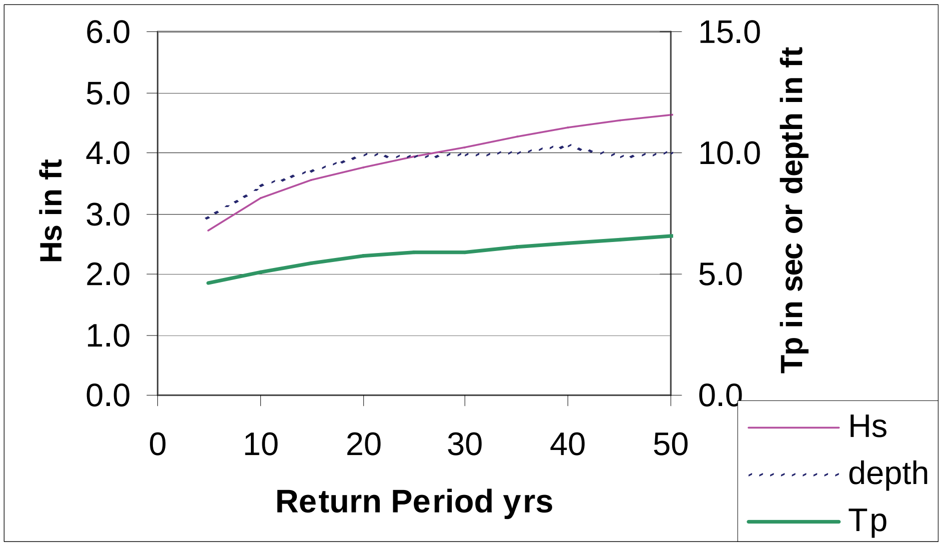

Figure H9. Barren Island sta 5 significant wave height, peak period, and depth (mllw) as function of return period (1 ft = 0.3048 m)

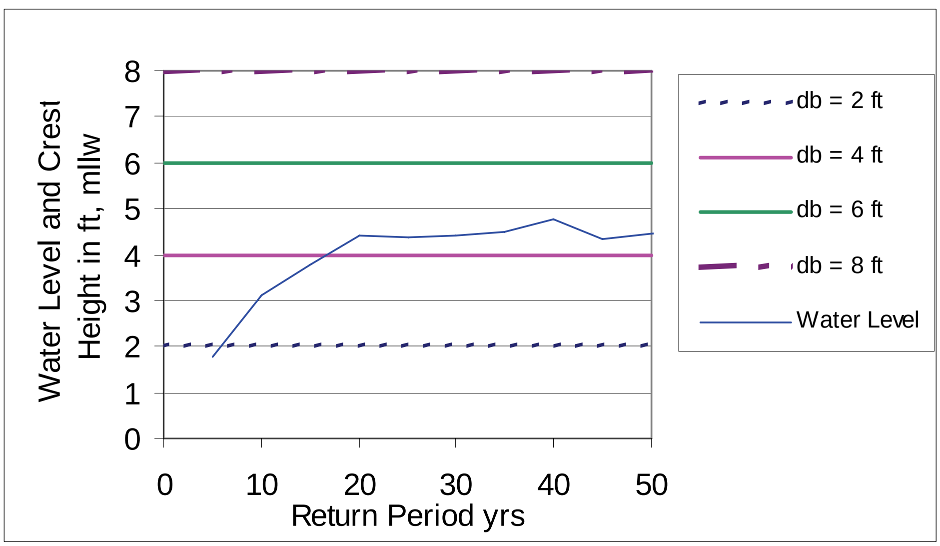

Figure H10. Barren Island sta 5 water level and crest height as function of return period (1 ft = 0.3048 m).

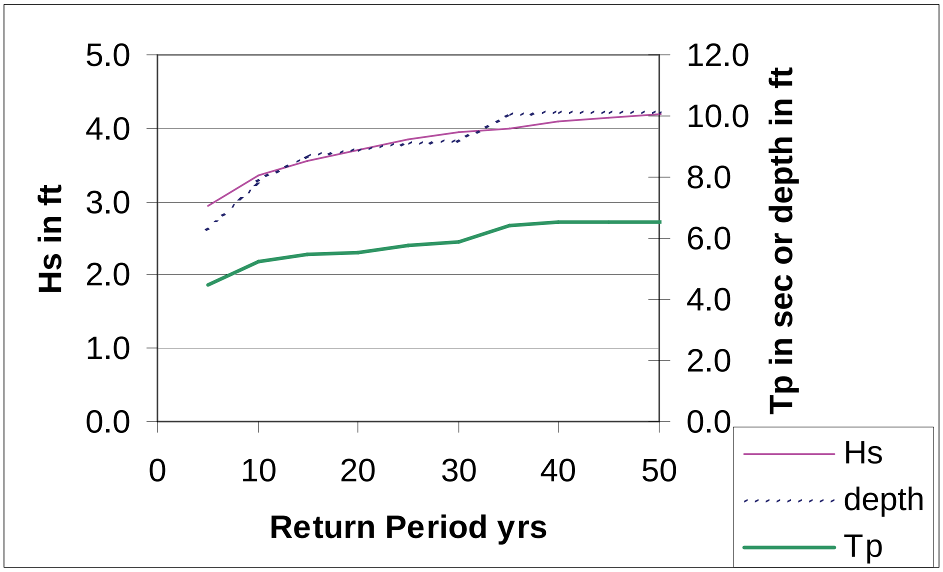

Figure H11. Barren Island sta 6 significant wave height, peak period, and depth (mllw) as function of return period (1 ft = 0.3048 m)

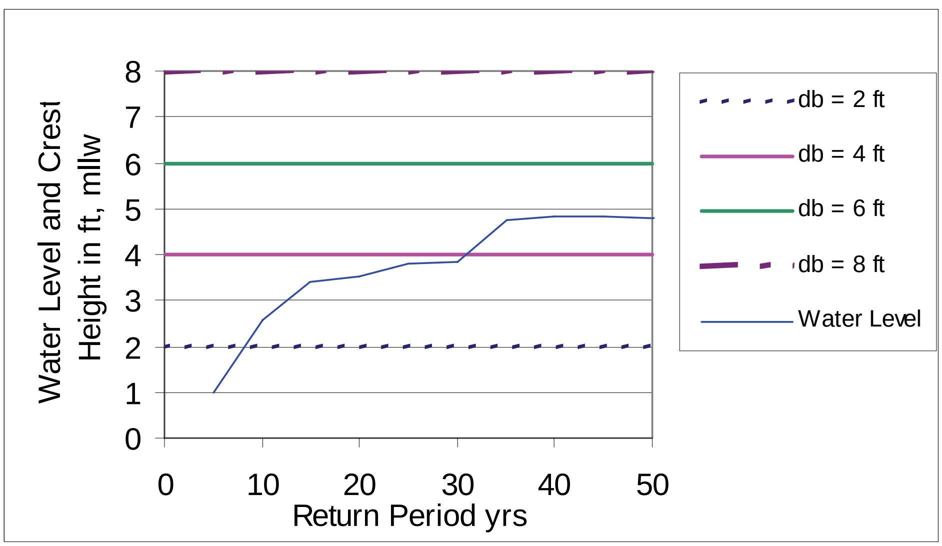

Figure H12. Barren Island sta 6 water level and crest height as function of return period (1 ft = 0.3048 m)

Table H7
Extreme Water Levels for Historical Northeasters from Barren Island Water Level Analysis, Station 2 (Figure 17)

Return Period in years	Water Level Relative to mllw in meters (ft)
5	0.68 (2.23)
10	0.79 (2.58)
25	0.92 (3.03)
50	1.02 (3.36)
100	1.12 (3.69)

Table H8
Extreme Water Levels for Historical Hurricanes from Barren Island Water Level Analysis Station 2 (Figure 17)

Return Period in years	Water Level Relative to mllw in meters (ft)
5	0.61 (1.99)
10	0.85 (2.79)
25	1.16 (3.80)
50	1.39 (4.55)
100	1.62 (5.30)

Appendix I
Armor Weight and Transmitted H_s as Function of Return Period for Barren Island

This appendix summarizes stable armor weight and stable underlayer weight as a function of return period for each design analysis station at Barren Island. For each station, crest freeboard heights of 0.61, 1.22, 1.83, and 2.44 m (2, 4, 6, and 8 ft) are shown. The stability relations of Melby and Hughes (2004) were used. Figures I1, I3, I5, I7, I9, and I11 show stable main armor weight as a function of return period for sta 1, 2, 3, 4, 5, and 6, respectively. Figures I2, I4, and I6 show wave transmission as a function of return period for sta 1, 2, and 3, respectively. The overtopping transmission is shown for crest freeboard heights of 0.61, 1.22, 1.83, and 2.44 m (2, 4, 6, and 8 ft). Figures I8, I10, and I12 show wave overtopping volume as a function of return period for sta 4, 5, and 8, respectively. The overtopping is shown for crest freeboard heights of 0.61, 1.22, 1.83, and 2.44 m (2, 4, 6, and 8 ft).

Table I1
Armor Weight as Function of Return Period for Station 1 with Crest Height of 2 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.66	4.66	0.0	180	18
10	2.79	4.66	0.0	222	22
15	2.79	4.66	0.0	233	23
20	2.79	4.66	0.0	239	24
25	2.79	4.66	0.0	245	25
30	2.79	4.66	0.0	246	25
35	2.79	4.66	0.0	252	25
40	2.79	4.66	0.0	257	26
45	2.79	4.66	0.0	263	26
50	2.79	4.66	0.0	265	27
100	2.79	4.66	0.0	265	27

Table I2
Armor Weight as Function of Return Period for Station 1 with Crest Height of 4 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.66	5.45	1.2	270	27
10	3.18	5.81	0.9	393	39
15	3.51	5.97	0.7	498	50
20	3.74	6.53	0.1	540	54
25	3.94	6.66	0.0	613	61
30	3.99	6.66	0.0	638	64
35	3.99	6.66	0.0	653	65
40	3.99	6.66	0.0	666	67
45	3.99	6.66	0.0	682	68
50	3.99	6.66	0.0	688	69
100	3.99	6.66	0.0	688	69

Table I3
Armor Weight as Function of Return Period for Station 1 with Crest Height of 6 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.66	5.45	3.2	363	36
10	3.18	5.81	2.9	626	63
15	3.51	5.97	2.7	794	79
20	3.74	6.53	2.1	817	82
25	3.94	7.09	1.6	838	84
30	4.13	7.09	1.6	939	94
35	4.30	7.41	1.2	991	99
40	4.43	7.55	1.1	1,058	106
45	4.53	7.68	1.0	1,114	111
50	4.63	7.94	0.7	1,142	114
100	5.18	8.63	0.0	1,380	138

Table I4
Armor Weight as Function of Return Period for Station 1 with Crest Height of 8 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.66	5.45	5.2	363	36
10	3.18	5.81	4.9	626	63
15	3.51	5.97	4.7	841	84
20	3.74	6.53	4.1	1,028	103
25	3.94	7.09	3.6	1,215	122
30	4.13	7.09	3.6	1,375	138
35	4.30	7.41	3.2	1,448	145
40	4.43	7.55	3.1	1,513	151
45	4.53	7.68	3.0	1,560	156
50	4.63	7.94	2.7	1,578	158
100	5.18	8.63	2.0	1,846	185

Table I5
Armor Weight as Function of Return Period for Station 2 with Crest Height of 2 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.99	5.58	0.0	252	25
10	3.35	5.58	0.0	370	37
15	3.35	5.58	0.0	376	38
20	3.35	5.58	0.0	389	39
25	3.35	5.58	0.0	391	39
30	3.35	5.58	0.0	407	41
35	3.35	5.58	0.0	418	42
40	3.35	5.58	0.0	420	42
45	3.35	5.58	0.0	427	43
50	3.35	5.58	0.0	428	43
100	3.35	5.58	0.0	447	45

Table I6
Armor Weight as Function of Return Period for Station 2 with Crest Height of 4 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.99	6.20	1.4	381	38
10	3.61	6.79	0.8	550	55
15	3.97	7.05	0.5	667	67
20	4.23	7.38	0.2	762	76
25	4.43	7.58	0.0	831	83
30	4.55	7.58	0.0	922	92
35	4.55	7.58	0.0	949	95
40	4.55	7.58	0.0	953	95
45	4.55	7.58	0.0	967	97
50	4.55	7.58	0.0	971	97
100	4.55	7.58	0.0	1,013	101

Table I7
Armor Weight as Function of Return Period for Station 2 with Crest Height of 6 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.99	6.20	3.4	501	50
10	3.61	6.79	2.8	888	89
15	3.97	7.05	2.5	1,028	103
20	4.23	7.38	2.2	1,121	112
25	4.43	7.74	1.8	1,169	117
30	4.63	7.97	1.6	1,273	127
35	4.79	8.33	1.2	1,336	134
40	4.95	8.46	1.1	1,426	143
45	5.09	8.69	0.9	1,491	149
50	5.22	8.86	0.7	1,559	156
100	5.65	9.42	0.2	1,846	185

Table I8
Armor Weight as Function of Return Period for Station 2 with Crest Height of 8 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.99	6.20	5.4	501	50
10	3.61	6.79	4.8	907	91
15	3.97	7.05	4.5	1,174	117
20	4.23	7.38	4.2	1,438	144
25	4.43	7.74	3.8	1,630	163
30	4.63	7.97	3.6	1,847	185
35	4.79	8.33	3.2	1,877	188
40	4.95	8.46	3.1	1,981	198
45	5.09	8.69	2.9	2,037	204
50	5.22	8.86	2.7	2,111	211
100	5.65	9.42	2.2	2,391	239

Table I9
Armor Weight as Function of Return Period for Station 3 with Crest Height of 2 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.76	5.48	0.0	209	21
10	3.29	5.48	0.0	346	35
15	3.29	5.48	0.0	360	36
20	3.29	5.48	0.0	373	37
25	3.29	5.48	0.0	387	39
30	3.29	5.48	0.0	391	39
35	3.29	5.48	0.0	396	40
40	3.29	5.48	0.0	399	40
45	3.29	5.48	0.0	409	41
50	3.29	5.48	0.0	410	41
100	3.29	5.48	0.0	433	43

Table I10
Armor Weight as Function of Return Period for Station 3 with Crest Height of 4 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.76	5.61	1.9	368	37
10	3.35	6.73	0.8	449	45
15	3.74	7.09	0.4	565	57
20	4.00	7.41	0.1	653	65
25	4.23	7.48	0.0	770	77
30	4.46	7.48	0.0	885	89
35	4.49	7.48	0.0	910	91
40	4.49	7.48	0.0	915	92
45	4.49	7.48	0.0	939	94
50	4.49	7.48	0.0	942	94
100	4.49	7.48	0.0	994	99

Table I11
Armor Weight as Function of Return Period for Station 3 with Crest Height of 6 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.76	5.61	3.9	411	41
10	3.35	6.73	2.8	738	74
15	3.74	7.09	2.4	878	88
20	4.00	7.41	2.1	966	97
25	4.23	7.84	1.6	1,039	104
30	4.46	8.07	1.4	1,140	114
35	4.63	8.20	1.3	1,228	123
40	4.76	8.27	1.2	1,305	131
45	4.89	8.43	1.0	1,388	139
50	4.99	8.53	0.9	1,441	144
100	5.51	9.19	0.3	1,785	179

Table I12
Armor Weight as Function of Return Period for Station 3 with Crest Height of 8 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.76	5.61	5.9	411	41
10	3.35	6.73	4.8	738	74
15	3.74	7.09	4.4	1,019	102
20	4.00	7.41	4.1	1,260	126
25	4.23	7.84	3.6	1,521	152
30	4.46	8.07	3.4	1,651	165
35	4.63	8.20	3.3	1,745	175
40	4.76	8.27	3.2	1,835	184
45	4.89	8.43	3.0	1,909	191
50	4.99	8.53	2.9	1,969	197
100	5.51	9.19	2.3	2,311	231

Table I13
Armor Weight as Function of Return Period for Station 4 with Crest Height of 2 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.85	6.46	0.0	231	23
10	3.51	6.46	0.0	422	42
15	3.87	6.46	0.0	560	56
20	3.88	6.46	0.0	591	59
25	3.88	6.46	0.0	596	60
30	3.88	6.46	0.0	600	60
35	3.88	6.46	0.0	602	60
40	3.88	6.46	0.0	604	60
45	3.88	6.46	0.0	607	61
50	3.88	6.46	0.0	607	61
100	3.88	6.46	0.0	647	65

Table I14
Armor Weight as Function of Return Period for Station 4 with Crest Height of 4 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.85	6.96	1.5	366	37
10	3.51	7.81	0.7	507	51
15	3.87	8.23	0.2	615	62
20	4.17	8.46	0.0	748	75
25	4.36	8.46	0.0	844	84
30	4.56	8.46	0.0	946	95
35	4.69	8.46	0.0	1,019	102
40	4.82	8.46	0.0	1,093	109
45	4.92	8.46	0.0	1,155	116
50	5.02	8.46	0.0	1,213	121
100	5.08	8.46	0.0	1,330	133

Table I15
Armor Weight as Function of Return Period for Station 4 with Crest Height of 6 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.85	6.96	3.5	456	46
10	3.51	7.81	2.7	833	83
15	3.87	8.23	2.2	940	94
20	4.17	8.79	1.7	1,023	102
25	4.36	8.56	1.9	1,185	118
30	4.56	8.92	1.5	1,243	124
35	4.69	8.89	1.6	1,337	134
40	4.82	8.79	1.7	1,448	145
45	4.92	9.15	1.3	1,446	145
50	5.02	9.15	1.3	1,514	151
100	5.71	9.78	0.7	2,002	200

Table I16
Armor Weight as Function of Return Period for Station 4 with Crest Height of 8 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.85	6.96	5.5	456	46
10	3.51	7.81	4.7	856	86
15	3.87	8.23	4.2	1,150	115
20	4.17	8.79	3.7	1,472	147
25	4.36	8.56	3.9	1,652	165
30	4.56	8.92	3.5	1,812	181
35	4.69	8.89	3.6	1,934	193
40	4.82	8.79	3.7	2,084	208
45	4.92	9.15	3.3	2,049	205
50	5.02	9.15	3.3	2,138	214
100	5.71	9.78	2.7	2,646	265

Table I17
Armor Weight as Function of Return Period for Station 5 with Crest Height of 2 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.72	7.28	0.2	234	23
10	3.25	7.51	0.0	367	37
15	3.54	7.51	0.0	473	47
20	3.77	7.51	0.0	575	58
25	3.94	7.51	0.0	645	65
30	4.10	7.51	0.0	717	72
35	4.27	7.51	0.0	805	81
40	4.43	7.51	0.0	902	90
45	4.51	7.51	0.0	953	95
50	4.51	7.51	0.0	964	96
100	4.51	7.51	0.0	1,014	101

Table I18
Armor Weight as Function of Return Period for Station 5 with Crest Height of 4 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.72	7.28	2.2	428	43
10	3.25	8.63	0.9	462	46
15	3.54	9.28	0.2	519	52
20	3.77	9.51	0.0	607	61
25	3.94	9.51	0.0	681	68
30	4.10	9.51	0.0	756	76
35	4.27	9.51	0.0	850	85
40	4.43	9.51	0.0	954	95
45	4.53	9.51	0.0	1,019	102
50	4.63	9.51	0.0	1,087	109
100	5.28	9.51	0.0	1,578	158

Table I19
Armor Weight as Function of Return Period for Station 5 with Crest Height of 6 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.72	7.28	4.2	429	43
10	3.25	8.63	2.9	737	74
15	3.54	9.28	2.2	800	80
20	3.77	9.91	1.6	832	83
25	3.94	9.88	1.6	927	93
30	4.10	9.94	1.6	1,009	101
35	4.27	10.01	1.5	1,107	111
40	4.43	10.27	1.2	1,182	118
45	4.53	9.84	1.7	1,331	133
50	4.63	9.97	1.5	1,384	148
100	5.28	10.76	0.8	1,789	179

Table I20
Armor Weight as Function of Return Period for Station 5 with Crest Height of 8 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.72	7.28	6.2	429	43
10	3.25	8.63	4.9	737	74
15	3.54	9.28	4.2	968	97
20	3.77	9.91	3.6	1,196	120
25	3.94	9.88	3.6	1,341	134
30	4.10	9.94	3.6	1,491	149
35	4.27	10.01	3.5	1,609	161
40	4.43	10.27	3.2	1,670	167
45	4.53	9.84	3.7	1,876	188
50	4.63	9.97	3.5	1,925	193
100	5.28	10.76	2.8	2,345	235

Table I21
Armor Weight as Function of Return Period for Station 6 with Crest Height of 2 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.95	6.30	1.0	338	34
10	3.35	7.31	0.0	400	40
15	3.54	7.31	0.0	475	48
20	3.71	7.31	0.0	533	53
25	3.84	7.31	0.0	597	60
30	3.94	7.31	0.0	644	64
35	4.00	7.31	0.0	711	71
40	4.10	7.31	0.0	762	76
45	4.13	7.31	0.0	777	78
50	4.20	7.31	0.0	807	81
100	4.39	7.31	0.0	963	96

Table I22
Armor Weight as Function of Return Period for Station 6 with Crest Height of 4 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.95	6.30	3.0	498	50
10	3.35	7.87	1.4	561	56
15	3.54	8.73	0.6	556	56
20	3.71	8.86	0.5	607	61
25	3.84	9.12	0.2	650	65
30	3.94	9.15	0.2	698	70
35	4.00	9.31	0.0	755	76
40	4.10	9.31	0.0	811	81
45	4.13	9.31	0.0	826	83
50	4.20	9.31	0.0	858	86
100	4.72	9.31	0.0	1,226	123

Table I23
Armor Weight as Function of Return Period for Station 6 with Crest Height of 6 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.95	6.30	5.0	498	50
10	3.35	7.87	3.4	795	80
15	3.54	8.73	2.6	862	86
20	3.71	8.86	2.5	924	92
25	3.84	9.12	2.2	958	96
30	3.94	9.15	2.2	1,013	101
35	4.00	10.07	1.2	944	94
40	4.10	10.14	1.2	997	100
45	4.13	10.14	1.2	1,016	102
50	4.20	10.10	1.2	1,058	106
100	4.72	11.22	0.1	1,303	130

Table I24
Armor Weight as Function of Return Period for Station 6 with Crest Height of 8 ft at Barren Island

Return Period, years	Significant Wave Height H_s, ft	Water Depth h, ft	Freeboard Rc, ft	Armor Stone Weight, lb	Underlayer Stone Weight, lb
5	2.95	6.30	7.0	498	50
10	3.35	7.87	5.4	795	80
15	3.54	8.73	4.6	965	97
20	3.71	8.86	4.5	1,085	109
25	3.84	9.12	4.2	1,226	123
30	3.94	9.15	4.2	1,326	133
35	4.00	10.07	3.2	1,351	135
40	4.10	10.14	3.2	1,410	141
45	4.13	10.14	3.2	1,433	143
50	4.20	10.10	3.2	1,489	149
100	4.72	11.22	2.1	1,707	171

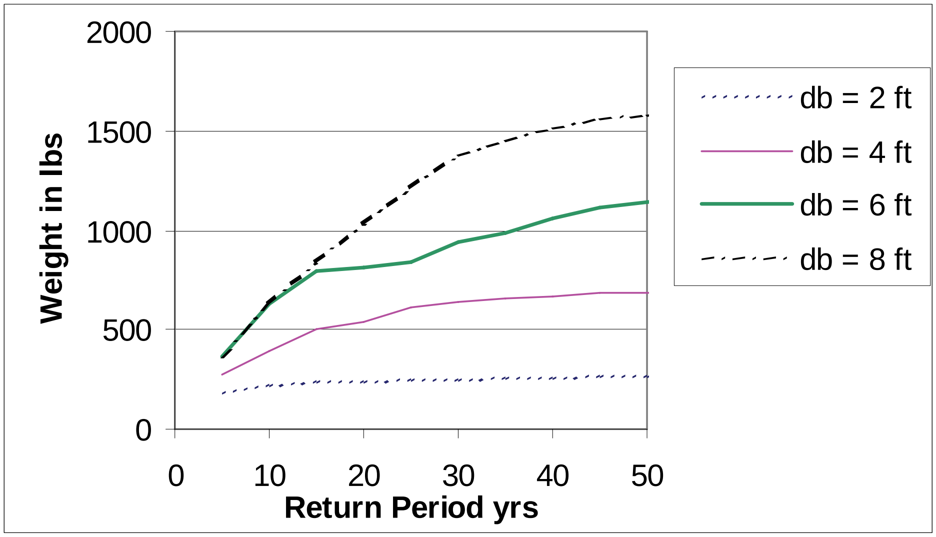

Figure I1. Barren Island sta 1 stable armor weight for various crest heights as function of return period

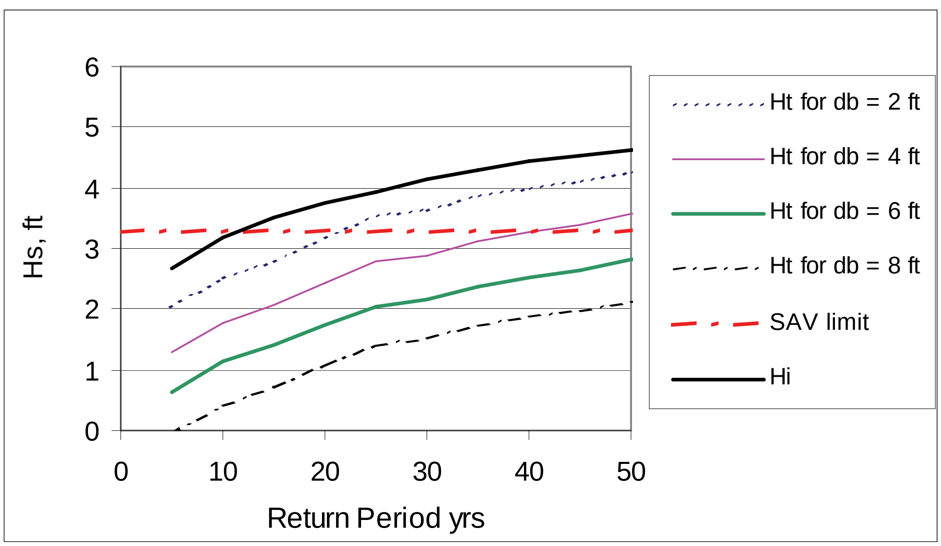

Figure I2. Barren Island sta 1 incident (Hi) and transmitted (Ht) significant wave heights for various crest heights as function of return period. Rough estimate of limiting wave height for submerged aquatic vegetation is also shown

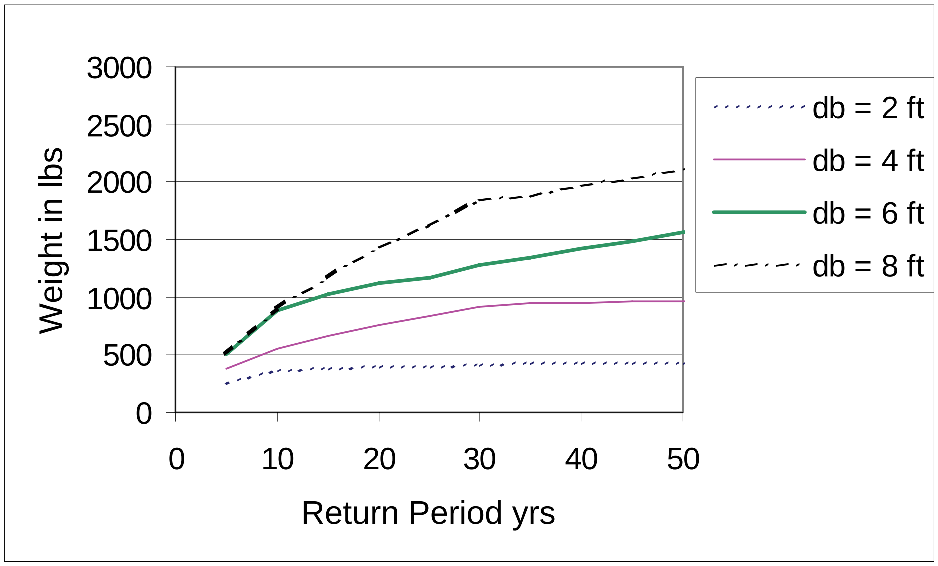

Figure I3. Barren Island sta 2 stable armor weight for various crest heights as function of return period

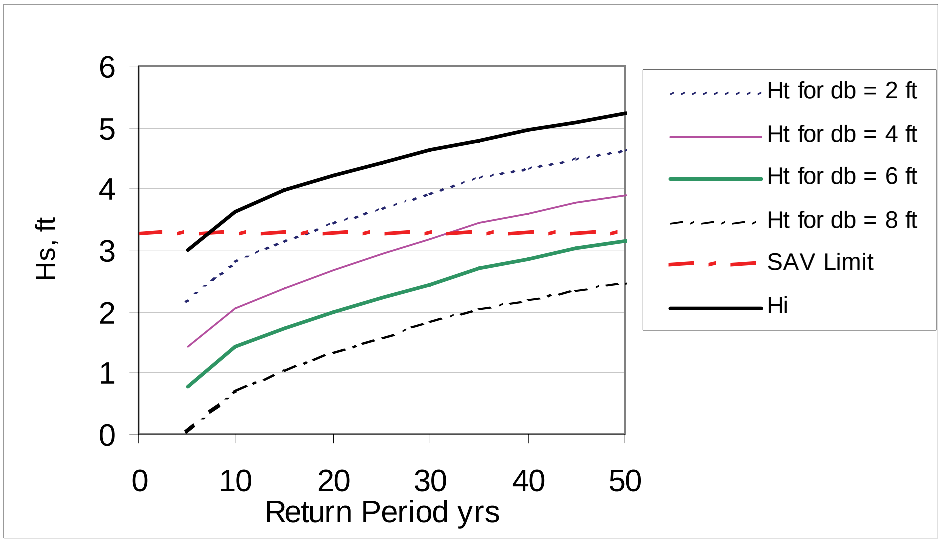

Figure I4. Barren Island sta 2 incident (Hi) and transmitted (Ht) significant wave heights for various crest heights as function of return period. Rough estimate of limiting wave height for submerged aquatic vegetation is also shown

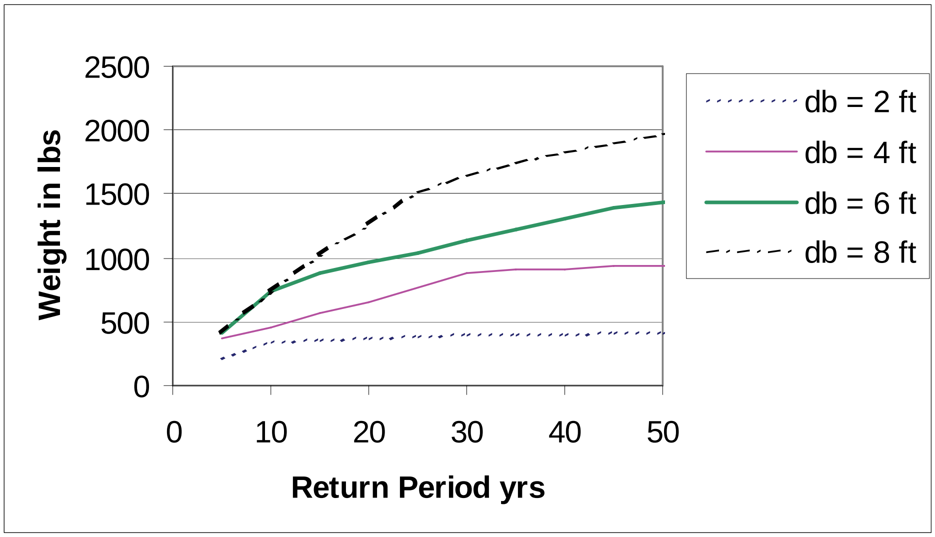

Figure I5. Barren Island sta 3 stable armor weight for various crest heights as function of return period

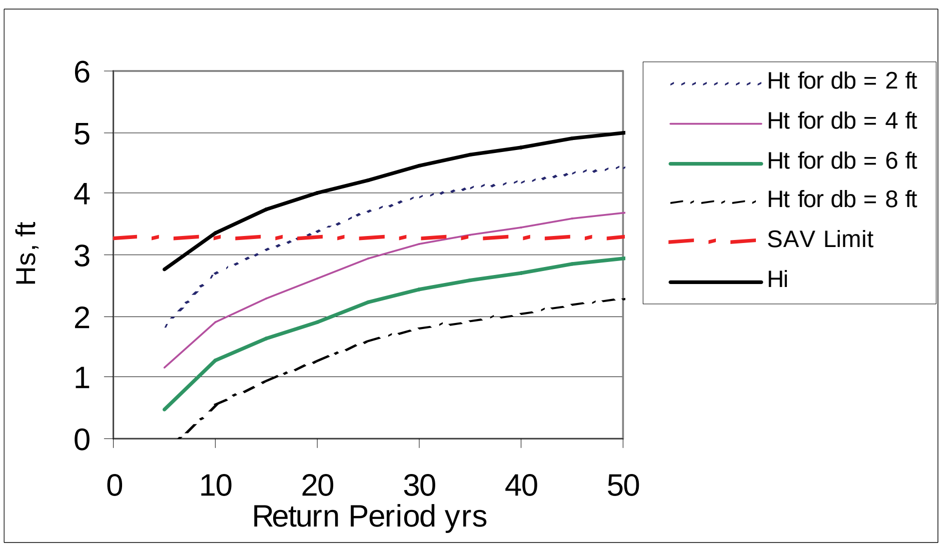

Figure I6. Barren Island sta 3 incident (Hi) and transmitted (Ht) significant wave heights for various crest heights as function of return period. A rough estimate of the limiting wave height for submerged aquatic vegetation is also shown

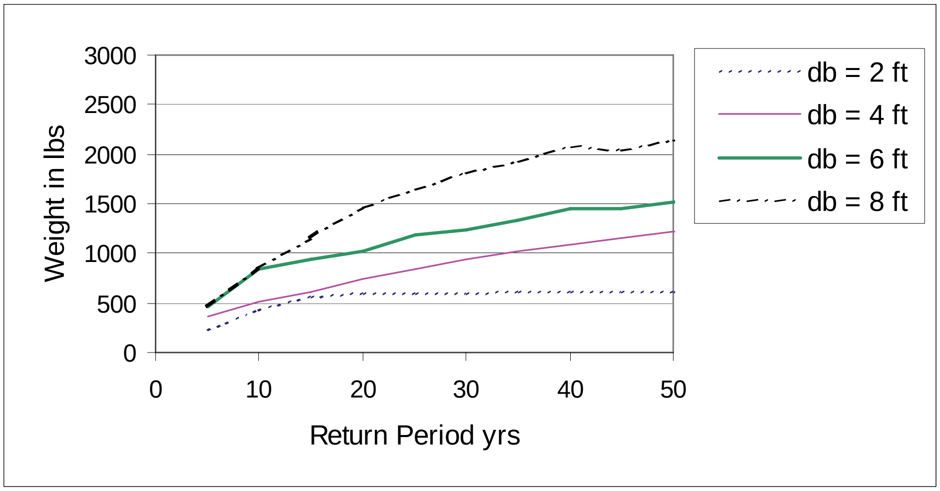

Figure I7. Barren Island sta 4 stable armor weight for various crest heights as function of return period

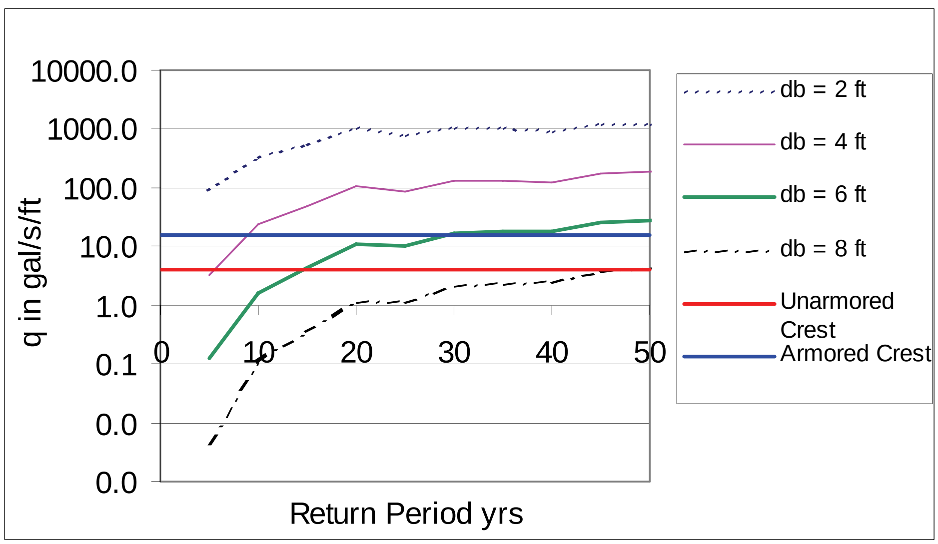

Figure I8. Barren Island sta 4 overtopping wave transmission for various crest heights as function of return period. Overtopping limits for unarmored and armored crests are also shown. 1 ft = 0.3048 m and 1 gal/sec/ft = 0.0124 cu m/sec/m

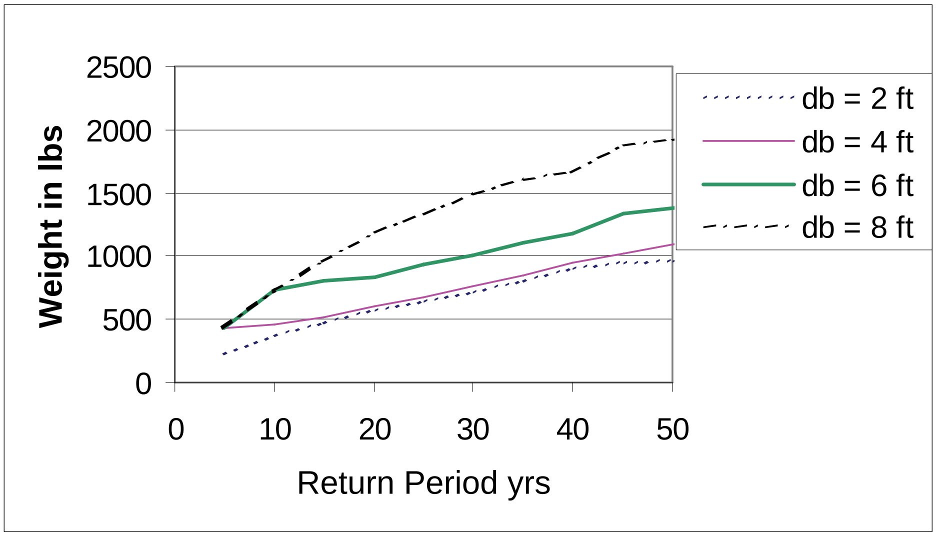

Figure I9. Barren Island sta 5 stable armor weight for various crest heights as function of return period

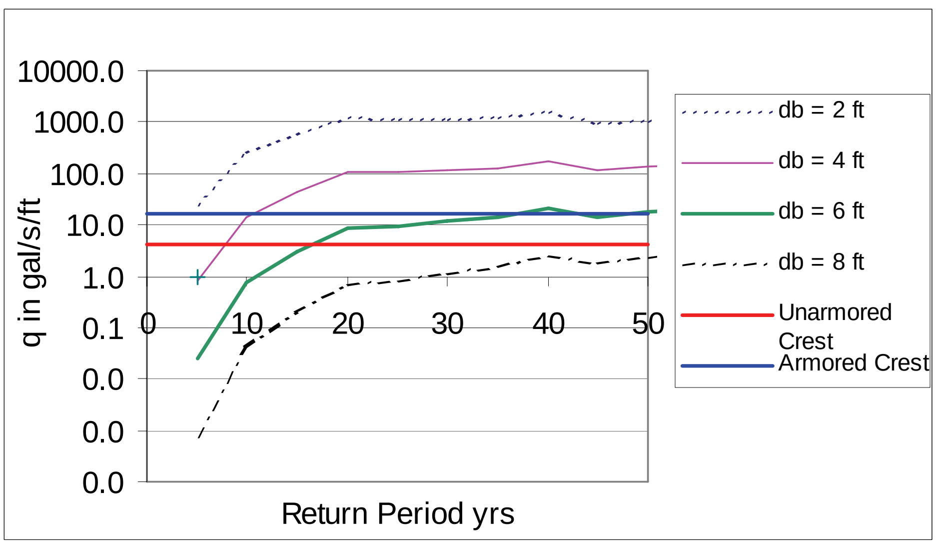

Figure I10. Barren Island sta 5 overtopping wave transmission for various crest heights as function of return period. Overtopping limits for unarmored and armored crests are also shown. 1 ft = 0.3048 m and 1 gal/sec/ft = 0.0124 cu m/sec/m

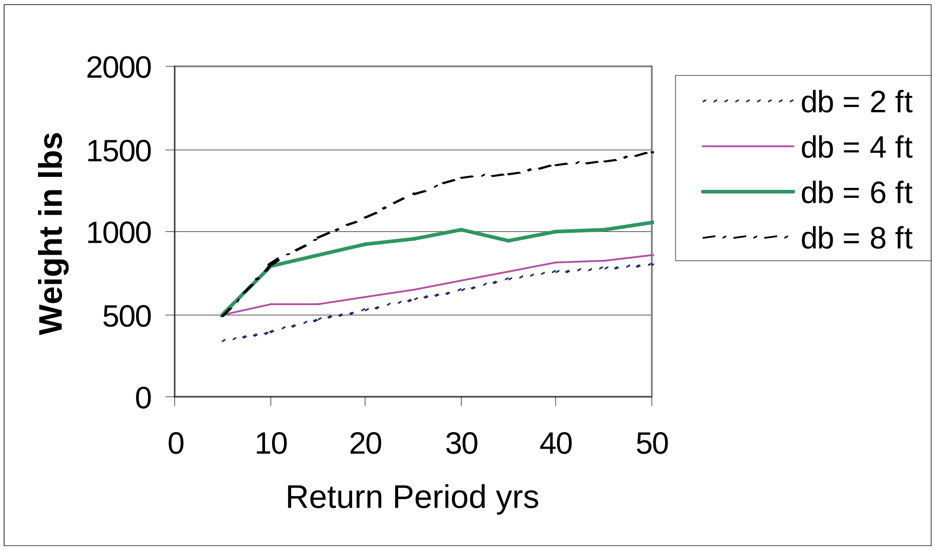

Figure I11. Barren Island sta 6 stable armor weight for various crest heights as function of return period

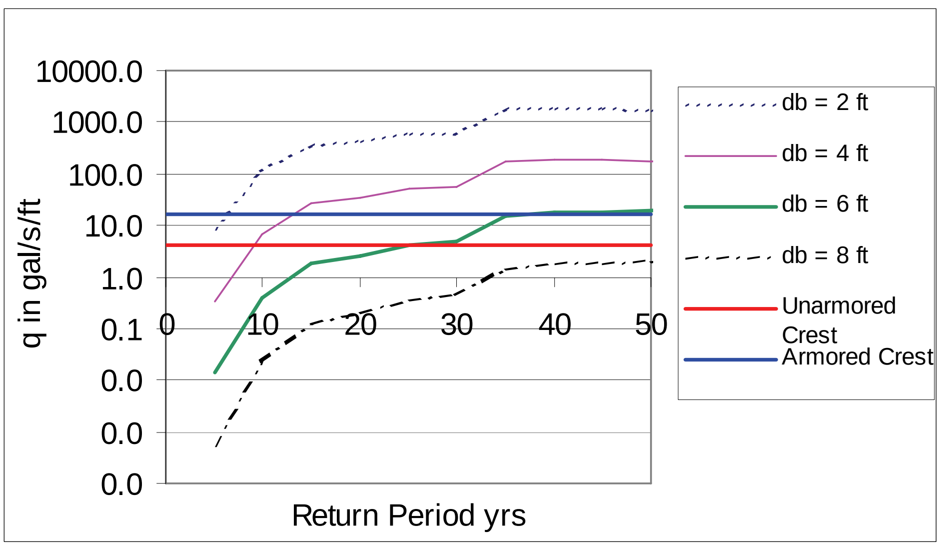

Figure I12. Barren Island sta 6 overtopping wave transmission for various crest heights as function of return period. Overtopping limits for unarmored and armored crests are also shown. 1 ft = 0.3048 m and 1 gal/sec/ft = 0.0124 cu m/sec/m

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14. ABSTRACT
This report summarizes the life-cycle design and optimization of structures on three islands in Chesapeake Bay. The islands are Poplar, James, and Barren. The life-cycle analysis is accomplished using a new method termed Empirical Life-Cycle Simulation (ELS). The historical storms selected for simulation include both winter storms (extratropical storms) and hurricanes (tropical storms). Historical water levels due to the combined effect of historical storms and astronomical tides are simulated using a numerical model of the entire Chesapeake Bay. A localized wind-wave growth model is used to hindcast historical waves. The waves are transformed to a number of analysis stations around each island using a separate numerical model. For each analysis location, 148-year time histories of waves and water levels at 3-hour intervals are produced for use in the life-cycle analysis phase of the study. A new empirical time series simulation method for waves and water levels is proposed so that the effects of potential future wave and water level climate can be analyzed. Finally, analysis of storm maximum and extremal analysis of waves and water levels is described for each island. The results of the wave and water level analyses and simulations described above are used to optimize the structure cross sections over the life cycle. Least-cost structural alternatives that also minimize maintenance requirements are proposed based on these investigations.

15. SUBJECT TERMS
See reverse.

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15. SUBJECT TERMS

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Chesapeake Bay	Storm surge
Design	STWAVE
Empirical simulation	Water levels
Life-cycle	Waves
Optimization	